SOME OF THE PHYSICAL PROPERTIES OF LIGHT WEIGHT AGGREGATE

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ABSTRACT

An expanded shale and an expanded clay light weight aggregate were used in tests to determine their physical properties in light weight concrete. The tests were paralleled using sand and gravel aggregate. Specific gravity and absorption of the aggregates, strength, durability and bonding properties of the concrete were determined.

Results indicated that these light weight aggregates could be used as concrete aggregates without sacrificing many of the physical properties of the concrete. The light weight concrete made using these two aggregates yielded superior results with regard to durability and bond.

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INTRODUCTION

Nature of Subject

The ever increasing demands of industry and the numerous structural problems which have arisen as a result of the increasing heights of buildings and length of bridge spans, as well as the frequent necessity of adding to existing structures where soil conditions may be a governing factor have led to the development of a new field in the building industry — that of light weight concrete. The unlimited possibilities of a building material that will give relatively light weight, structural strength, heat and sound insulation, durability, chemical inertness, fireproofness and imperviousness to moisture have in recent years brought it to the attention of engineers and architects.

This newly awakened interest in the light weight concrete field has manifested itself in the growing need for structural and physical properties of the light weight aggregate being used to produce this new concrete.

Light weight aggregates are generally classified into two chief categories:

(1) ultra light weight aggregate

(2) light weight aggregate

The ultra light weight aggregates include materials weighing about twenty pounds per cubic foot or less. This category includes exfoliated vermiculite, expanded perlite and pumicite and other similar materials. Light weight aggregates are also cellular but heavier than the ultra light weight particle. The light weight aggregate includes materials that weigh from 40 to 60 pounds per cubic foot. This is considerably lighter than sand and gravel which weigh upwards of 100 pounds per cubic foot.

Light weight aggregates may be produced from a variety of materials. Possible sources of raw materials may be classified as natural, by-product, or manufactured aggregates. Natural aggregates include clinker, which is an expanded material formed by the burning of coal beds, and scoria, a volcanic product. By-product aggregates include cinders, a residue from high-temperature combustion of coal in industrial plants; sintered fly ash; and artificially expanded blast-furnace slags. Manufactured aggregates include artificially expanded clays, shales and slates.

Of these light weight aggregates those made by using an expanded clay, shale or slate have proven generally superior, and as a result considerable research has been done to establish the structural and physical properties of these light weight aggregates. Expanded clays, shales or slates have been recognized by the building industry for over a third of a century although it is only in the last decade that they have achieved the outstanding prominence they now occupy.

History of Subject

The process was first perfected at Kansas City in 1917 by Stephen J. Hayde, a chemist, who found that by heating certain clays, shales or slates to incipient fusion ($1900^\circ - 2200^\circ$ F.) the oxidation of its carbon content formed gases which expanded producing myriads of tiny air cells within the mass which were retained upon cooling and solidification. The resultant product was a highly cellular aggregate commonly called Haydite after its first producer.

In commercial production the materials are heated in this same range ($1900^\circ - 2200^\circ$ F.). Usually this is done in a rotary kiln although sintering is used in some processes. The "bloating property" as it is commonly called

does not depend on the basic type of clay minerals present as equally good bloating clays may be found among both kaolinite and montmorillinite type clays. Clays and shales generally used for making bloated products are very similar to those used in brick, tile and other structurally manufactured clay products. They are the common or low-grade clays with fusibility ranging from 1900° - 2200° F. which expand satisfactorily on heating.

Strangely enough it is not the clay or shale components which provide the bloating qualities but the extraneous material usually referred to as "impurities". These materials give the clay or shale its bloating characteristics generating the gas or vapor which causes the expansion of the material when it is heated to incipient fusion and is in a semi-plastic state.

The two general commercial processes involved in producing light weight aggregate are rotary-kiln firing or a sintering process. In the rotary-kiln process, which is by far the most common, the raw material is first crushed, in the case of shale or slate, or pelletized or extruded in the case of clay, whereupon it is screened to a limiting coarse size. The material freed of deleterious material, such as rock, is then transferred to rotary kilns which may be fired with either natural gas, fuel oil or pulverized coal. The bloated product is then taken from the kilns, cooled, crushed and screened to size for shipment.

The other process, which is that of sintering, is similar. In the sintering operation crushed coal is mixed with the crushed raw material and pelletized before firing. Other than this the process is the same as that of the rotary kiln. The bloated product is then cooled, crushed and screened to size.

The determining factor as to which process is used is the temperature

range over which the material will bloat. Materials with large bloating temperature ranges are generally more suitable for rotary-kiln processing. Although there are exceptions, depending on the material used, this temperature range should exceed 200° F. As the bloating-temperature range decreases, the chance for agglomeration or sticking in a rotary kiln increases and the firing becomes extremely difficult and critical. Materials suitable for rotary-kiln firing and also those that have too narrow bloating-range temperatures are often suitable for sintering. (11)*

In connection with the tests conducted throughout this investigation only the products of rotary-kiln processes were used. One was a product of shale of established quality from a large manufacturer in the United States, and the other of clay - a local product. Throughout the investigation the tests of Haydite concrete were paralleled by tests of sand and gravel concrete to furnish comparisons of the properties found and to give an idea of the uniformity of the materials, test methods and workmanship employed.

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* Numbers in parentheses refer to the list of references in the bibliography.

Chapter I

General Layout of Testing Program

The intention of this program was to carry out tests which would furnish information regarding the structural and physical properties of concrete made from light weight aggregates.

To accomplish this purpose, pours of mix designs with water-cement ratios of 0.4; 0.5; 0.6 and 0.7 were made. In each instance 12 - 3" diameter by 6" cylinders were cast. These cylinders were broken in the following sequence - 2 cylinders at 7 days; 2 cylinders at 21 days; 4 cylinders at 28 days; 2 cylinders at 35 days and 2 cylinders at 42 days.

During the pouring of the mixes the workability of the concrete made with the light weight aggregates was observed to be inferior to that of the sand and gravel mixes. The mixes were then repeated with the design based on air-entrainment. A common air-entraining agent, "Darex", was used.

Four beams - 16" x $4\frac{1}{2}$" x $3\frac{1}{2}$" - were made up as part of each pour. Two of these beams were used as freeze-thaw specimens (See Appendix II), and two were used to obtain the modulus of rupture for the material.

The next series of tests were run to obtain values of bond between the steel and the light weight concrete. To accomplish this, pull-out specimens were made. Four w/c ratios and two types of reinforcing rods - plain and hibond* - were used. The rods were 1"; 3/4"; 5/8" and 1/2" in diameter. In the 5/8" diameter series the ratio of bearing to shearing area was changed by using a different hibond rod.

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* The term hibond refers to the deformed bars used throughout this report which meet A.S.T.M. specification C 330.

Chapter II

Materials and Material Tests

The materials used in the tests conducted were of two kinds:

(1) Sand and gravel aggregate.

(2) Haydite aggregate.

The sand and gravel used were those supplied commercially by the O. K. Construction and Supply Company, Edmonton. Two types of haydite aggregate were used - an expanded clay and an expanded shale. The expanded clay product was supplied by the Lite Rock Division of the Edmonton Concrete Block Company Limited, Edmonton, and is referred to as Russell's aggregate. The expanded shale is a Smithwick Concrete Products aggregate and was supplied by Precast Concrete Limited, Edmonton. It is referred to as Smithwick's aggregate.

Coarse Aggregate Tests

The physical properties and the grading of the coarse aggregates used are given in Table 1. The maximum size of all aggregates was that portion passing a 1" screen.

TABLE 1

Physical Properties of Coarse Aggregate

Material	Absorption 24 hr. basis % by weight	Dry Unit weight #/cu. ft.	Bulk Specific Gravity
Gravel	0.6	105.5	2.62
Smithwick's 3/4" - 3/8"	14.6 13.8	36.8 36.4	1.21 1.26
Smithwick's 3/8" - 3/16"	18.4 18.1	43.6 43.9	1.71 1.94
Russell's 3/4" - 3/16"	17.5 17.1	34.3 34.1	0.96 1.01

Grading of Coarse Aggregate

Material	% Retained on Screen as Indicated			
	3/4"	3/8"	#4	Pan
Gravel	22.6	62.2	15.2	–
Smithwick's 3/4" – 3/8"	1.3	46.9	39.6	12.2
Smithwick's 3/8" – 3/16"	–	–	27.4	72.6
Russell's 3/4" – 3/16"	2.2	26.3	60.5	11.0

Fine Aggregate Tests

The physical properties and grading of the fine aggregates used are listed in Table 2.

TABLE 2

Physical Properties of Fine Aggregates

Physical Properties

Material	Absorption % by weight	Dry Unit weight #/cu. ft.	Color Test	Specific Gravity*
Sand	1.1	106.7	#2	2.67
Smithwick's	16.2	45.7	#1	2.23
	15.8	45.5	#1	2.19
	15.4			2.10
	15.5			2.10
Russell's	18.3	44.1	#1	2.08
	18.5	43.8	#1	2.15
	18.1			2.02
	18.3			2.07

* Bulk specific gravity (saturated-surface dry basis).

Grading

Material	% Retained on Screen Indicated								
	#4	#8	#16	#30	#50	#100	#200	Silt	F.M
Sand	10.2	4.4	6.9	17.1	44.7	14.0	1.3	1.4	2.66
Smithwick's	13.3	22.8	15.1	8.5	5.9	5.9	7.6	20.9	2.97
	10.0	25.5	17.1	8.5	5.8	5.8	7.2	20.1	2.99
Russell's	28.5	26.5	18.6	9.4	6.1	4.4	4.0	2.8	4.21
	21.0	27.5	21.8	11.6	7.7	5.2	3.4	1.8	4.06

The mortar-making properties of the fine aggregates were determined by mortar cube tests made in accordance with A.S.T.M. specification C87. The results follow in Table 3.

TABLE 3

Mortar Making Properties

Material	Compressive Strength p.s.i.*		
	3 day	7 day	28 day
Standard Ottawa Sand	1170	1415	2920
Sand	1330	1585	3490
Smithwick's	1375	1830	3740
Russell's	1360	1675	3570

* Average of 3 cubes.

Specific Gravity

The results of specific gravity tests are of greater importance with haydite aggregate than with sand and gravel aggregate. As can be seen, the consistency and repeatability of the determinations of specific gravity and absorption never completely approach the accuracy that is obtained from sand and gravel. Much of this is due to the difference in surface of the

aggregates. The haydite types of aggregate have open porous surfaces which do not readily lend themselves to true determination of a saturated surface dry condition.

Initially, specific gravities were not run according to A.S.T.M. specification C - 128. The aggregate was allowed to soak for a period of time greater than 24 hours. Some of the tests were run after a week of soaking. The results are shown in Table 2. For the Smithwick fine aggregate, the range of values was 2.10 to 2.23. Russell's fine aggregate had a range from 2.02 to 2.15. The trend indicated that the longer the period of soaking, the higher the value obtained for specific gravity.

Richart and Jensen [1] at Illinois found that the various size fractions - #4, #8, #16, etc. - had different specific gravities. This could explain the variance of results obtained. With this in mind, a series of specific gravities were run on the size fractions. A.S.T.M. specification C - 128 was followed. The results are listed in Table 4.

TABLE 4

Specific Gravities of Size Fractions

Sieve Size	Specific Gravities: Smithwick Aggregate, Run 1	Smithwick Aggregate, Run 2	Russell Aggregate, Run 1	Russell Aggregate, Run 2
# 4	1.55	1.59	1.54	1.58
# 8	1.75	1.73	1.65	1.81
# 16	1.93	1.84	1.98	1.89
# 28	2.16	2.06	2.14	2.07
# 50	2.30	2.13	2.29	2.21
#100	2.32	2.33	2.51	2.38
pan	2.46	2.50	2.50	2.41

Once more the reproducibility of results is not good. At best it would appear to go back to the statement – "the consistency and repeatability of the determination of specific gravity and absorption never completely approach the accuracy that is obtained from sand and gravel."

The values of specific gravity used for the mix designs were obtained by arbitrarily averaging the available results. This gave a sufficiently accurate value to start with.

Cement Tests

The cement used was standard Portland cement made at the Canada Cement plant at Exshaw, Alberta. The cement was obtained in three lots and tested as outlined by the Canadian Standards Association. The results of these tests follow in Table 5. Specifications were met in all instances.

TABLE 5

Cement Tests

	Compressive Strength – p.s.i.*		
	3 day	7 day	28 day
Standard	900	1800	3000
Lot #1	1284	2031	3891
Lot #2	1250	2130	3415
Lot #3	1189	2074	3760

	Tensile Strength – p.s.i.*		
	3 day	7 day	28 day
Standard	150	275	350
Lot #1	161	294	376
Lot #2	158	286	365
Lot #3	167	304	394

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* Average of 3 specimens.

	Time of Set - (Gillmore Test)	
	Initial Set	Final Set
Standard	1 hr.	8 hrs.
Lot #1	4 hrs. 13 min.	7 hrs. 32 min.
Lot #2	4 hrs. 8 min.	7 hrs. 15 min.
Lot #3	4 hrs. 15 min.	7 hrs. 35 min.

The mixing water used was taken directly from the Edmonton City water main. The air entraining agent used in all instances was "Darex" - manufactured by the Dewey and Almy Chemical Company, Cambridge, Mass.

Chapter III

Mix Designs, Specimen Manufacture and Curing

The American Concrete Institute method was employed in the design of all mixes. The table and adjustment of values used were taken from the Journal of the American Concrete Institute, November 1943 (see Table 15, Appendix III). A new method of design was proposed in the Journal of the American Concrete Institute, October 1954, which supersedes the method used but unfortunately this took place after the tests for this report had been completed. The design of a sand and gravel mix with a w/c ratio of 0.40 follows as an example.

Conditions:

w/c = 0.40 slump = 3"

Maximum size of coarse aggregate = 3/4"

F.M. of sand = 2.66

Design (basis of 1 cubic yard):

(From table) Sand Content = 46.0%

Adjustments –

$$\text{w/c} \quad \frac{-(0.57 - 0.40) \times 1}{0.05} = -3.4\%$$

$$\text{F.M.} \quad \frac{-(2.75 - 2.66) \times 0.5}{0.01} = -0.5\%$$

Net Sand Content = 42.1%

(From table) Water Content = 310#/cu. yd.

$$\text{Cement Content} = \frac{310}{0.4} = 775\text{\#/cu. yd.}$$

Computing on Absolute Volumes:

$$\text{Water} = \frac{310}{62.4} = 4.96 \text{ cu. ft.}$$

$$\text{Cement} = \frac{775}{62.4 \times 3.15} = 3.94 \text{ cu. ft.}$$

Total Volume of Paste = 8.90 cu. ft.

Absolute Volume of Aggregate = 27.00 – 8.90 = 18.10 cu. ft.

$$\text{Absolute Volume of Sand} = \frac{42.1}{100} \times 18.10 = 7.62 \text{ cu. ft.}$$

Absolute Volume of Coarse Aggregate = 10.48 cu. ft.

Weight of Aggregates

Weight of Sand = 7.62 x 62.4 x 2.67 = 1270 #/cu. yd.

Weight of Coarse Aggregate = 10.48 x 62.4 x 2.62 = 1715 #/cu. yd.

Proportions for 1 cubic yard
(Saturated surface dry aggregate)

Cement	=	775 #
Water	=	310 #
Sand	=	1270 #
Coarse Aggregate	=	1715 #

The mix proportions used were based on 1.15 cubic feet and were for this case –

Cement	=	33.0 #
Water	=	11.6 #
Sand	=	55.7 #
Coarse Aggregate	=	73.0 #

The design of the air-entrained mixes was based on Table 16, Appendix III. As mentioned before the air-entraining agent used was "Darex". The initial

quantities used were determined from Figure 57, Appendix III.[6] These values were adjusted when necessary.

In the case of light weight mixes one variation from the above method was employed. In all cases the combined aggregates in their proper proportions were mixed with sufficient water to place them in a saturated surface dry condition prior to mixing with the cement and water. A 24 hour soaking period allowed the aggregate to reach this condition.

In all cases 3" diameter x 6" cylinder molds were used. This size cylinder mold was used because of the large number of cylinders (12) being cast from each batch. Had the normal size cylinder been used it would have necessitated the mixing of large quantities of concrete involving several batches to a pour. In the case of the cylinders, because of their small size, a 5/16" diameter tamping rod was used instead of the standard 5/8" diameter rod. The concrete was put in three layers with 25 roddings to each layer.

The four beams cast with each pour were 16" x 4½" x 3½" in dimension. Their size permitted the use of the standard tamping rod. They were cast in three layers and rodded 25 times per layer. The surface was lightly trowelled to present a smooth surface on four sides for the freeze-thaw tests.

The strength cylinders and beams were left in the molds for a 24 hour period to "set up". In some cases it was impossible to strip the molds from the light weight concrete for at least 36 hours.

All strength test cylinders and freeze-thaw beams were cured in the moist room of the University Civil Engineering Department. The moist room is kept at 70° F. with 100% humidity as specified in A.S.T.M. standards. This standard has since been superseded by Supplement #3 1954. The strength

cylinders were broken at specified times; however, this was not true of the freeze-thaw beams.

Initially, the beams were put into the apparatus after a definite number of days of curing. Because of the limited capacity of the freeze-thaw apparatus and the length of time necessary to complete the standard 300 cycles, this was not always possible. As a result, the time of curing varied with the availability of space in the freeze thaw apparatus.

The pull-out specimens were made in the following way. The forms were cubicle in size being 12 times the diameter of the rod. Six specimens were made for each w/c ratio and for each size of rod. Three of the rods were hibond and three were plain. In making the specimens the rods were maintained vertical in the center of the forms and the concrete tamped in around them in three layers with 25 roddings per layer. The exposed surface was trowelled lightly and the specimens left to "set up". The sand and gravel mixes were stripped the following day. In most cases the light weight concrete required an extra day before the forms could be stripped.

Because of the large number of specimens, it was impossible to cure them in the moist room. To compensate for this and insure proper curing, the blocks were piled and canvas hoses placed across the tops of them. This insured sufficient moisture and since the room was maintained at approximately 70° F. the curing conditions closely approached A.S.T.M. standard.

Chapter IV

Observations on Concrete Mixes

In Table 6 the mix proportions, slump, water added where necessary, workability, finishability, rodability and any comments on the ordinary (non-air-entrained) mixes are tabulated.

The sand and gravel mixes in all cases were rich looking, well proportioned with good workability. In each case a slight bit of water had to be added to bring the mix to the desired slump. This may be accounted for by the coarse aggregate which was comparatively dirty.

The Smithwick aggregate produced a light weight concrete with workability which was only slightly inferior to the comparable sand and gravel mixes. In the 0.7 w/c mix, there was slight evidence of bleeding. The mixes became a bit harsher with increasing w/c ratios. This is ordinarily expected.

The light weight concrete made by using Russell's aggregate was definitely inferior to that of the sand and gravel mixes as well as those made with Smithwick's aggregate. In all cases the mixes were harsh, becoming harsher with increasing w/c ratios. There was a tendency for the coarse aggregate to float in the 0.6 and 0.7 w/c ratio mixes. These two mixes were poured at slumps of 4" and 3½" respectively which may partially account for this. There was also evidence of bleeding in all cases. The bleeding was slight in the low w/c ratios (0.4 and 0.5) increasing to where it was serious in the higher w/c ratios.

Because of the harshness of the mixes and the tendency to bleed as well as segregate, the mixes were repeated using an air-entrained mix design (See Table 7). The most noticeable improvement due to the addition of the air

TABLE 6

Observations on Ordinary Concrete Mixes

W/C (i)	Cement lbs.	Water lbs.	Fine Agg. lbs.	Coarse Agg. lbs.	Water added lbs.	Slump ins.	Air content % (ii)	Workability	Finishability	Rodability	Comments
Sand & Gravel											
0.4	33.0	11.6	55.7	73.0	2.2	3	1.5	Ex.	Ex.	V.G.	Rich looking. Some surface cracks when forms stripped.
0.5	26.4	11.8	60.8	73.2	1.0	3	2.0	Ex.	Ex.	V.G.	Rich looking.
0.6	22.0	11.8	65.4	72.5	1.0	3	2.0	G.	G.	G.	Well proportioned.
0.7	18.9	11.1	69.6	69.3	2.5	3	2.2	G.	G.	G.	Well proportioned.
Russell's Aggregate											
0.4	35.0	14.0	52.2	21.7	—	3	6.0	F.	F.	G.	Harsh mix. Evidence of bleeding.
0.5	28.5	14.2	57.0	22.9	1.0	3	6.0	F.	G.	G.	Mix quite harsh.
0.6	24.1	14.5	56.4	23.1	-1.0	4	6.0	F.	F.	F.	Some segregation and bleeding. Coarse agg. tended to float.
0.7	20.3	14.0	56.9	21.5	—	3½	5.5	P.	P.	F.	Same as 0.6
Smithwick Aggregate											
0.4	35.1	14.1	50.8	*12.7 20.5	3.0	3½	5.0	G.	G.	F.	Quite nice mix.
0.5	27.2	13.6	55.0	*13.2 19.6	4.0	3½	5.5	G.	G.	F.	Same as 0.4
0.6	22.0	13.2	52.3	*12.4 18.4	—	3½	4.5	G.	G.	F.	Same as 0.4
0.7	18.2	12.8	52.6	*11.4 17.1	—	2½	5.0	G.	F.	F.-P.	Mix a bit harsh. Slight evidence of bleeding.

* indicates intermediate agg. Ex. = excellent V.G. = very good G. = good F. = fair P. = poor

(i) Aggregate in saturated surface dry condition. (ii) As recorded on pressure meter.

was the marked increase in workability. In all cases, the mixes looked less harsh, exhibited more cohesion, were devoid of any evidence of bleeding and generally segregated less than the ordinary mixes.

The sand and gravel mixes looked rich as a result of the air entrainment. They finished well, were cohesive and sticky. Even in the high w/c ratios the mixes looked rich and well proportioned.

The concrete made using Smithwick's aggregate still appeared a bit harsh but approached sand and gravel standards. As with the sand and gravel, the mixes were much more cohesive and showed no signs of segregation. No difficulty was encountered in entraining approximately the same amount of air in all of the mixes. Where it was necessary to add water to the mix a check of the moisture content of the aggregate revealed it low, thus accounting for the necessary additions.

The concrete made using Russell's aggregate was still quite harsh. The effect of air entrainment was not enough to offset the poor gradation of the fines. It improved the mix considerably in the low w/c ratios but could not do away with the segregation which appeared in the 0.6 and 0.7 w/c ratio mixes. Upon stripping the molds, a honeycombed structure was often revealed. This was also evident in the ordinary mixes made with Russell's aggregate. Considerable difficulty was encountered in entraining the air in these mixes. The values jumped over a comparatively wide range. This was possibly caused by change in gradation of the fines from mix to mix.

A look at the values listed for air contents of the light weight concrete mixes reveals abnormally high values. A check of the values listed in Table 6 shows values ranging from 4.5% to 6.0% entrained air without the addition of an air entraining agent. In all cases, a pressure type air meter was used

TABLE 7

Observations on Air-Entrained Concrete Mixes

W/C[i]	Cement lbs.	Water lbs.	Fine Agg. lbs.	Coarse Agg. lbs.	Water added lbs.	Slump ins.	Darex c.c.	Air Content % [ii]	Workability	Finishability	Rodability	Comments
Sand & Gravel												
0.4	26.6	10.7	47.4	74.2	2.2	3	8.3	$3\frac{1}{2}$	Ex.	Ex.	G.	Plastic, cohesive mix. Very well proportioned.
0.5	25.7	12.8	60.5	87.5	–	3	10.0	$4\frac{1}{2}$	Ex.	Ex.	G.	" " " " " "
0.6	21.7	13.1	64.7	86.0	–	$2\frac{1}{2}$	10.0	4	G.	G.	G.	" " " " " "
0.7	19.0	13.4	68.6	84.0	–	3	10.0	6	G.	G.	G.	Mix rich and well proportioned.
Russell's Aggregate												
0.4	29.6	11.8	43.1	24.2	–	$3\frac{1}{4}$	9.5	12	G.	F.–G.	F.	Mix somewhat harsh. No segregation.
0.5	23.2	11.6	44.7	23.2	–1.1	3	9.2	14	G.	F.	F.	No segregation or bleeding. Somewhat harsh in appearance.
0.6	19.6	11.8	47.6	22.7	–2.7	3 3/4	8.4	11	G.–F.	F.	F.	Harsh mix. Cohesive and workable. Evidence of segregation.
0.7	17.0	12.0	50.2	22.1	–2.1	3 3/4	8.3	15	G.	G.	F.	Mix harsh yet workable and cohesive. Slight amount of segregation.
Smithwick Aggregate												
0.4	27.0	10.8	38.7 *14.7	23.6	2.1	$3\frac{1}{2}$	8.3	9	G.	G.	F.	Cohesive mix. No segregation or bleeding. Looks better than non-entrained mixes.
0.5	22.1	11.0	41.7 *14.6	23.5	1.6	3	9.0	$9\frac{1}{2}$	G.	G.	F.	Improved due to addition of air.
0.6	18.7	11.2	44.5 *14.4	23.1	–	$2\frac{1}{2}$	9.5	10	G.	G.	F.	Mix cohesive. No segregation. Looked better than 0.5 mix probably due to more air.
0.7	16.4	11.4	47.0 *14.1	22.5	–	$3\frac{1}{2}$	9.5	10	G.	F.	F.	Mix cohesive. No segregation. Looked harsh.

* indicates intermediate aggregate. Ex. = excellent V.G. = Very Good G = Good F = Fair P = Poor

(i) Aggregate in saturated surface dry condition. (ii) As recorded on pressure meter.

and this the author feels, is the answer. It is generally accepted that pressure meters force water into the voids of light weight aggregate when the concrete is being tested. The air in the voids must either compress or leave the aggregate when water is forced in. This air is entrapped in the cement slurry and gives an indication of excessive entrained air. This, however, was not known prior to the use of the pressure type air meter.

Upon the use of the pressure-type air meter an initial air content reading ranging from 4.5% to 6.0% was registered. This value gradually rose to from 9.5% to 11.0% in a matter of a minute or two. The pressureair meter works on the application of Boyle's law and usually has a working pressure of 15 p.s.i. This explains the gradual creep of the air content value. The reduction of the working pressure of the meter results in a slower forcing of air into the voids of the aggregate and the resultant gradual creep. To make sure that the air was not in the cement slurry, an air content was run on the aggregate alone. The aggregates in their proper proportions, after 24 hours of soaking, were put into the meter and the voids filled with water as in the air content test for concrete. The initial air content registered as before was approximately 6.0% creeping to 11.0%. An unsuccessful attempt was made to try and waterproof the aggregate and see if this would prevent the forcing of air into the voids. The result of the tests seemed to indicate that light weight concrete would register an air content of approximately 6.0% without an air entraining agent being used when a pressure-type air meter was used to determine the air contents.

Using this value of 6.0% as an initial air content, Darex was added to give air content readings in a range varying from 9.0%-12.0%. It was felt that this would be equivalent to a range of 3.0% - 6.0% entrained air in sand

Observations on Pull-Out Specimen Mixes

W/C (i)	1" Slump ins.	1" Air Content % (ii)	3/4" Slump ins.	3/4" Air Content % (ii)	5/8" Slump ins.	5/8" Air Content % (ii)	1/2" Slump ins.	1/2" Air Content % (ii)	
Sand & Gravel									
0.4	2½	4½ 5	2½	2½	2½	3½	2½	4	Mixes were rich and workable.
0.5	3	5½ 5	2½	8	2½	4	3	5	Appeared well proportioned. No
0.6	3	6 6	2½	5	3	5	3	5½	segregation or bleeding.
0.7	3	7 7	2½	5	3	4½	2½	4½	Finishability very good.
Russell's Aggregate									
0.4	3	9 10	2	10	2	11	1	12	Mixes were harsh and hard to
0.5	3	9 10	2	10	2	11	3½	12	handle. Left honeycombed surface
0.6	4	20 20	1½	12	2	11	2	12	when forms stripped. Lack of fines
0.7	4	15 15	1½	12	3	15	3	12	is very apparent. No segregation or bleeding evident.
Smithwick Aggregate									
0.4	3½	9 9	3	8	2½	10	1	9	Mixes finished quite well and
0.5	2½	10 10	1	7	2½	10	3	11	generally looked good. No apparent
0.6	3½	9 9	3	9	2½	11	3	12	segregation or bleeding. Stripping
0.7	3	10 10	3	11	2½	10	3	12	of forms revealed only slight amount of honeycombing. Air contents were quite easy to control.

(i) Aggregate in saturated surface dry condition. (ii) As recorded on pressure meter.

and gravel concrete. In some instances air contents as high as 15.0% were registered. As explained before, this was very possibly due to the gradation of the fines. Since the obtaining of the pressure meter used to measure air contents throughout these tests, it has been found that concerns using light weight concrete in the United States prefer to use a "Roll-A-Meter" (Charles R. Watts Co., Seattle, Washington). This instrument is similar to an oversized pycnometer and operates without air pressure, thus giving a true air content.

In addition to the preceding mixes, a series of mixes were poured for pull-out bond tests. It was decided to pour these mixes simulating as closely as possible actual conditions on a job. As a result the mixes were poured on a slump design basis. That is, the water was added to give the required slump regardless of the amount. As would generally be experienced under job conditions the light weight aggregate was pre-wetted to a moisture content of approximately 10.0% - 12.0%. The slump used as a criterion was from 2" to 2-1/2". The same mix designs were used as for the strength tests with the aforementioned variation - that of water addition. Table 8 gives the slump, air content and any observations and comments on the mixes as they were poured.

Conclusions

The observations on the ordinary concrete mixes (Table 6) indicate:

1) Light weight concrete is harsher than sand and gravel concrete.

2) Comparable slumps of light weight concrete and sand and gravel concrete do not indicate the same consistency.

3) Light weight aggregate segregates badly.

The harshness of light weight concrete mixes is due primarily to the shape of the particle. The two light weight fine aggregates used were produced by crushing. As a result the particles are angular and cubicle in shape. Add to this the fact that the particle is expanded and contains myriads of tiny air cells which when crushed present a rough surface and you have the major cause of harshness in light weight concrete mixes.

Gradation of light weight fines has a marked effect on the harshness of the mix because of this uneven and rough surface texture. This is brought out clearly by the two light weight fine aggregates used. The F.M. (fineness modulus) of Russell's fine aggregate was 4.20. This aggregate showed the harshest mixes with evidence of segregation and bleeding. Smithwick's fine aggregate had a F.M. of 2.98 and showed less harshness and next to no segregation and bleeding.

The sand particles by comparison are round and smooth. As a result the harshness which light weight concrete possesses is not apparent in sand and gravel mixes.

The light weight coarse aggregate has an uneven surface due to bloating. However, since all light weight coarse aggregate particles are not crushed they present a coated surface which should not detract from the workability of the mixes.

Comparable slumps of light weight concrete and sand and gravel concrete do not indicate the same consistency. Ordinarily in the case of concrete made with Smithwick's aggregate a slump of 1½" to 2" was equivalent to a sand and gravel mix slump of 3". As a rule, for a comparable consistency, light weight concrete would have a slump of 1½" to 2" less than its sand and gravel counterpart. Slump tests on concrete made from Russell's aggregate

frequently indicated no slump; yet the cement and water paste would drain and run from the cone of concrete.

Another point which deserves discussion is the segregation of the light weight aggregate prior to use. The aggregate, despite care in handling, if dry will tend to segregate into its various size constituents. It will also dust badly thereby losing much needed fines. The prime method of combatting this segregation is by pre-wetting the aggregate. Pre-wetting does away with dusting and tends to minimize the segregation of the particles. To combat this further, most manufacturers distribute light weight aggregate in a greater number of ~~and smaller~~ grading ranges. This is particularly true above the #4 size. The ranges are usually 3/8" - 3/16" or #4 and 3/4" - 3/8" and so on.

The observations on the air-entrained concrete mixes indicate that air-entrainment is necessary to increase the workability. The increase in workability was such that only large reductions in strength due to air-entrainment could rule it out as an integral part of light weight concrete.

Chapter V

Strength Test Results

The results of strength tests are found in Table 9. The cylinders as noted before were 3" in diameter and 6" high. In marking the various cylinders the following notations were used:

S – Smithwick's aggregate

R – Russell's aggregate

A – 7 day test

B – 21 day test

C – 28 day test

D – 35 day test

E – 42 day test

4 – w/c = 0.4

5 – w/c = 0.5

6 – w/c = 0.6

7 – w/c = 0.7

1 – first series of tests

An "A" preceding all other symbols denotes an air-entrained mix.

An example is R4A1. The R denotes Russell's aggregate was used in the mix; the 4 indicates the design w/c ratio of 0.4; the A indicates a 7 day test while the 1 indicates the first series of tests. Had an A preceded the notation-AR4A1 it would indicate an air-entrained mix.

As stated before, two cylinders constituted the results for any particular test. This is by no means a complete enough test but the enormity

of the tests undertaken limited the number of cylinders in each test to this number. A comprehensive report was desired rather than statistically accurate test results.

The cylinder results are in good agreement when one considers the following factors:

(1) Segregation of aggregate when handling.

(2) Segregation in mixes.

(3) Bleeding in most light weight non-entrained mixes.

(4) Frequent honeycombing of cylinders due to harshness of mix.

(5) Coefficient of variation of two cylinders.

Most of these factors were greatly alleviated by air entraining and show up in much closer agreement of the two cylinder results from the air-entrained mixes.

The strength results are presented in two ways. Figures 1 to 4 and 6 to 9 are graphs of "strength" vs "age" for the various w/c ratios and the various aggregates. These are the "w/c" vs "strength" curves for the ordinary and air-entrained mixes of the various aggregates. A close examination of these graphs indicates that air entrainment substantially decreased the strengths of the concretes. It also shows that the sand and gravel concrete mixes had the better strengths closely followed by the Smithwick aggregate concrete and then Russell's aggregate concrete. It was decided to plot "cement content" vs "strength" for the ordinary and air-entrained mixes. These plots are shown in Figures 11 to 13. Figure 14 is a compilation of Figures 11 to 13.

The results of the tests of this investigation as noted before are shown in two different manners. First they are shown as "w/c" vs "compressive strength" curves. Using a w/c ratio in connection with light weight concrete

is extremely difficult as one is unable to compute or measure the absorption taking place during the mixing period unless the aggregate is in a saturated surface dry condition. As a result it was realized that w/c ratios, although still holding true, meant very little when dealing with light weight concrete. For this reason the second set of graphs is presented in which "cement content" in bags is graphed against "compressive strength". This gives a much better picture of strengths obtained and is in good agreement with the method of control used in light weight concrete.

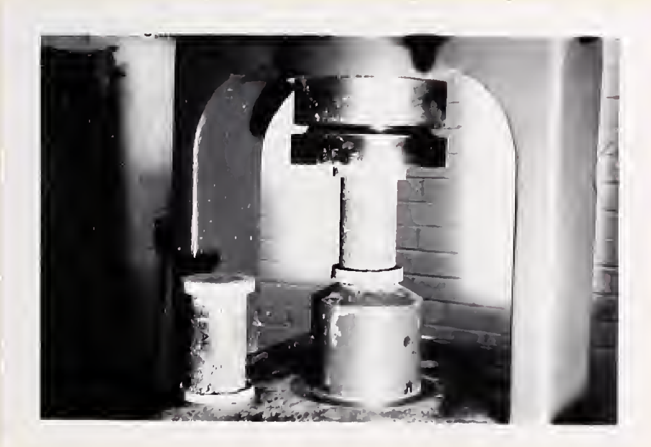
The "cement content" vs "strength" graphs show comparable strengths for the air-entrained and ordinary mixes as is normally expected from well controlled air-entrained mixes. This then definitely makes air-entrainment an integral part of light weight concrete. The increased workability without loss of strength necessitates its inclusion at all times.

The shape of the "cement content" vs "strength" graphs for the ordinary and air-entrained mixes also deserves some discussion. In all cases for the air-entrained mixes the curves became steeper with increasing cement contents, whereas the curves became flatter for the ordinary mixes (see Fig. 11-13). This would seem to indicate that in the higher cement content range the admixture imparted increased strengths to the mix. This is directly opposite to generally accepted results.[12, 18, 19]

The air entrainment of the light weight mixes increased their workability. This was true of the high as well as the low cement content mixes. The increase in workability and the accompanying decrease in mixing water in the high cement content mixes may explain the increasing slope at high cement factors. No explanation for the increased slopes of the air-entrained sand and gravel mixes is apparent. All sand and gravel mixes possessed good

workability before the air-entrainment.

A consideration must be made of the strengths obtained from the light weight concrete as compared to those of sand and gravel. In all cases the sand and gravel concrete exhibited slightly higher strengths. The strengths obtained using Smithwick's aggregates were only a slight bit lower than those obtained from using sand and gravel. Russell's aggregate gave results comparatively lower than the rest. This was due to poor gradation of the fines which caused bleeding and segregation in the mixes. The comparison of the Smithwick's aggregate and the sand and gravel aggregate shows that comparable strengths can be obtained using these aggregates and using cement factors of the same order. This applies only to a well graded light weight aggregate.



Photograph No. 1 – Breaking Concrete Test Cylinder

TABLE 9

Strength Test Cylinder Results (Ordinary Mixes)

Cylinder No.	Age - days	U.C.L.*	U.C.S.**
4A1	7	19300	2730
	7	20800	2940
R4A1	7	13500	1910
	7	17600	2490
S4A1	7	21500	3040
	7	23400	3310
5A1	7	24300	3440
	7	23200	3285
R5A1	7	11000	1560
	7	11500	1630
S5A1	7	15700	2220
	7	17750	2510
6A1	7	14500	2055
	7	14800	2095
R6A1	7	6000	0850
	7	6000	0850
S6A1	7	10600	1500
	7	10600	1500
7A1	7	12500	1770
	7	9400	1330
R7A1	7	5800	0820
	7	5400	0765
S7A1	7	10600	1500
	7	10300	1460
4B1	21	27450	3890
	21	31850	4570
R4B1	21	22200	3140
	21	23400	3310
S4B1	21	30000	4250
	21	31300	4430

Cylinder No.	Age - days	U.C.L.*	U.C.S.**
5B1	21	30400	4310
	21	31600	4475
R5B1	21	15000	2125
	21	16000	2265
S5B1	21	27350	3870
	21	25650	3640
6B1	21	24600	3480
	21	26850	3800
R6B1	21	9400	1330
	21	8500	1205
S6B1	21	17900	2530
	21	17500	2480
7B1	21	21350	3020
	21	21150	2995
R7B1	21	6800	0965
	21	-	-
S7B1	21	15700	2225
	21	14400	2040
4C1	28	40800	5780
	28	37200	5270
R4C1	28	26500	3755
	28	25400	3600
S4C1	28	30000	4250
	28	31000	4390
5C1	28	33400	4730
	28	35400	5010
R5C1	28	15200	2150
	28	17600	2490
S5C1	28	23400	3325
	28	23600	3345
6C1	28	26000	3680
	28	21800	3085
R6C1	28	10000	1420
	28	10100	1430

Cylinder No.	Age – days	U.C.L. *	U.C.S. **
S6C1	28	18600	2635
	28	19000	2690
7C1	28	24000	3400
	28	24000	3400
R7C1	28	8600	1220
	28	8200	1160
S7C1	28	16600	2350
	28	14200	2010
4D1	35	40200	5690
	35	36600	5180
R4D1	35	27200	3850
	35	23600	3340
S4D1	35	29850	4225
	35	35300	5000
5D1	35	35600	5040
	35	31400	4450
R5D1	35	23200	3285
	35	18250	2580
S5D1	35	29500	4180
	35	27700	3925
6D1	35	28000	3965
	35	26300	3720
R6D1	35	11550	1635
	35	15000	2120
S6D1	35	21700	3075
	35	21100	2990
7D1	35	20900	2960
	35	22400	3170
R7D1	35	10300	1460
	35	11300	1600
S7D1	35	23000	3260
	35	20500	2910
4E1	42	39000	5525
	42	34400	4875

Cylinder No.	Age – days	U.C.L. *	U.C.S. **
R4E1	42	26300	3725
	42	27300	3865
S4E1	42	38300	5420
	42	35300	5000
5E1	42	32800	4650
	42	32400	4590
R5E1	42	20400	2890
	42	21600	3060
S5E1	42	30450	4320
	42	29000	4110
6E1	42	29200	4140
	42	26500	3760
R6E1	42	12500	1770
	42	14900	2120
S6E1	42	23100	3270
	42	22400	3170
7E1	42	24300	3440
	42	23300	3300
R7E1	42	8100	1150
	42	8000	1130
S7E1	42	19500	2760
	42	20100	2850

TABLE 9 (cont'd.)

Strength Test Cylinder Results (Air-entrained Mixes)

Cylinder No.	Age - days	U.C.L. *	U.C.S. **
A4A1	7	24300	3440
	7	23300	3300
AR4A1	7	13600	1925
	7	11900	1685
AS4A1	7	15400	2180
	7	14600	2075
A5A1	7	14100	1995
	7	15000	2125
AR5A1	7	4700	0665
	7	4300	0610
AS5A1	7	7900	1120
	7	7300	1035
A6A1	7	15500	2195
	7	14300	2025
AR6A1	7	4800	0680
	7	3600	0510
AS6A1	7	5700	0808
	7	5700	0808
A7A1	7	9000	1275
	7	8400	1190
AR7A1	7	3000	0425
	7	3400	0480
AS7A1	7	5750	0815
	7	5750	0815
A4B1	21	28750	4070
	21	30000	4250
AR4B1	21	16700	2365
	21	16800	2380
AS4B1	21	20200	2860
	21	19500	2760

Cylinder No.	Age - days	U.C.L. *	U.C.S. **
A5B1	21	22000	3115
	21	23200	3285
AR5B1	21	10100	1430
	21	10500	1490
AS5B1	21	15000	2125
	21	16000	2265
A6B1	21	16500	2335
	21	15300	2165
AR6B1	21	9000	1275
	21	8800	1250
AS6B1	21	11500	1630
	21	10500	1490
A7B1	21	13400	1900
	21	15600	2210
AR7B1	21	5000	0708
	21	5300	0750
AS7B1	21	10600	1500
	21	10750	1520
A4C1	28	25800	3650
	28	31200	4420
AR4C1	28	17800	2520
	28	18400	2600
AS4C1	28	24600	3480
	28	26600	3780
A5C1	28	23800	3360
	28	24500	3460
AR5C1	28	12000	1700
	28	12000	1700
AS5C1	28	18800	2660
	28	18000	2540
A6C1	28	28000	4110
	28	32100	4550
AR6C1	28	8100	1150
	28	8050	1140

Cylinder No.	Age - days	U.C.L.*	U.C.S.**
AS6C1	28	13400	1900
	28	9400	1330
A7C1	28	7700	1190
	28	14400	2040
AR7C1	28	5800	0820
	28	6000	0850
AS7C1	28	10100	1430
	28	9700	1370
A4D1	35	32800	4650
	35	35500	5030
AR4D1	35	18500	2620
	35	19000	2690
AS4D1	35	31400	4450
	35	31400	4450
A5D1	35	25900	3670
	35	29500	4180
AR5D1	35	10600	1500
	35	11800	1670
AS5D1	35	17350	2460
	35	18400	2605
A6D1	35	23100	3270
	35	19900	2820
AR6D1	35	9000	1275
	35	10300	1460
AS6D1	35	15250	2175
	35	14000	1985
A7D1	35	18000	2550
	35	21250	3010
AR7D1	35	6200	0879
	35	5800	0821
AS7D1	35	12700	1800
	35	15100	2140
A4E1	42	42300	6000
	42	41000	5810

Cylinder No.	Age - days	U.C.L.*	U.C.S.**
AR4E1	42	19000	2690
	42	21800	3090
AS4E1	42	28700	4070
	42	29900	4240
A5E1	42	22400	3170
	42	29100	4120
AR5E1	42	13550	1920
	42	13500	1915
AS5E1	42	22900	3245
	42	22100	3130
A6E1	42	21300	3020
	42	24300	3440
AR6E1	42	10400	1470
	42	10450	1480
AS6E1	42	15850	2245
	42	16500	2340
A7E1	42	20500	2900
	42	20000	2830
AR7E1	42	8200	1160
	42	7500	1060
AS7E1	42	14500	2055
	42	16100	2280

* Ultimate compressive load - lbs.

** Ultimate compressive strength - p.s.i.

TABLE 10

Cement Contents and Strengths for Air-entrained and Ordinary Mixes

W/C ratios	Cement Contents bags/cu. yd.		Strengths - p.s.i. *	
	Ordinary	Air-entrained	Ordinary	Air-entrained
Sand & Gravel				
0.4	8.85	7.46	5235	5650
0.5	7.09	6.10	4615	3340
0.6	5.81	5.16	3940	3220
0.7	5.06	4.52	3370	2865
Russell's Aggregate				
0.4	9.40	8.00	3820	2890
0.5	7.98	6.51	2975	1915
0.6	6.72	5.50	1940	1475
0.7	5.86	4.76	1140	1110
Smithwick's Aggregate				
0.4	9.43	7.58	4760	4145
0.5	7.65	6.19	4210	3185
0.6	6.47	5.25	3220	2295
0.7	5.62	4.58	2800	2165

* Average of two cylinders.

[illegible]

				[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
				[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
				[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[illegible]

FIG. I

AGE - STRENGTH CURVE - W/C= 0.40 ORDINARY MIXES

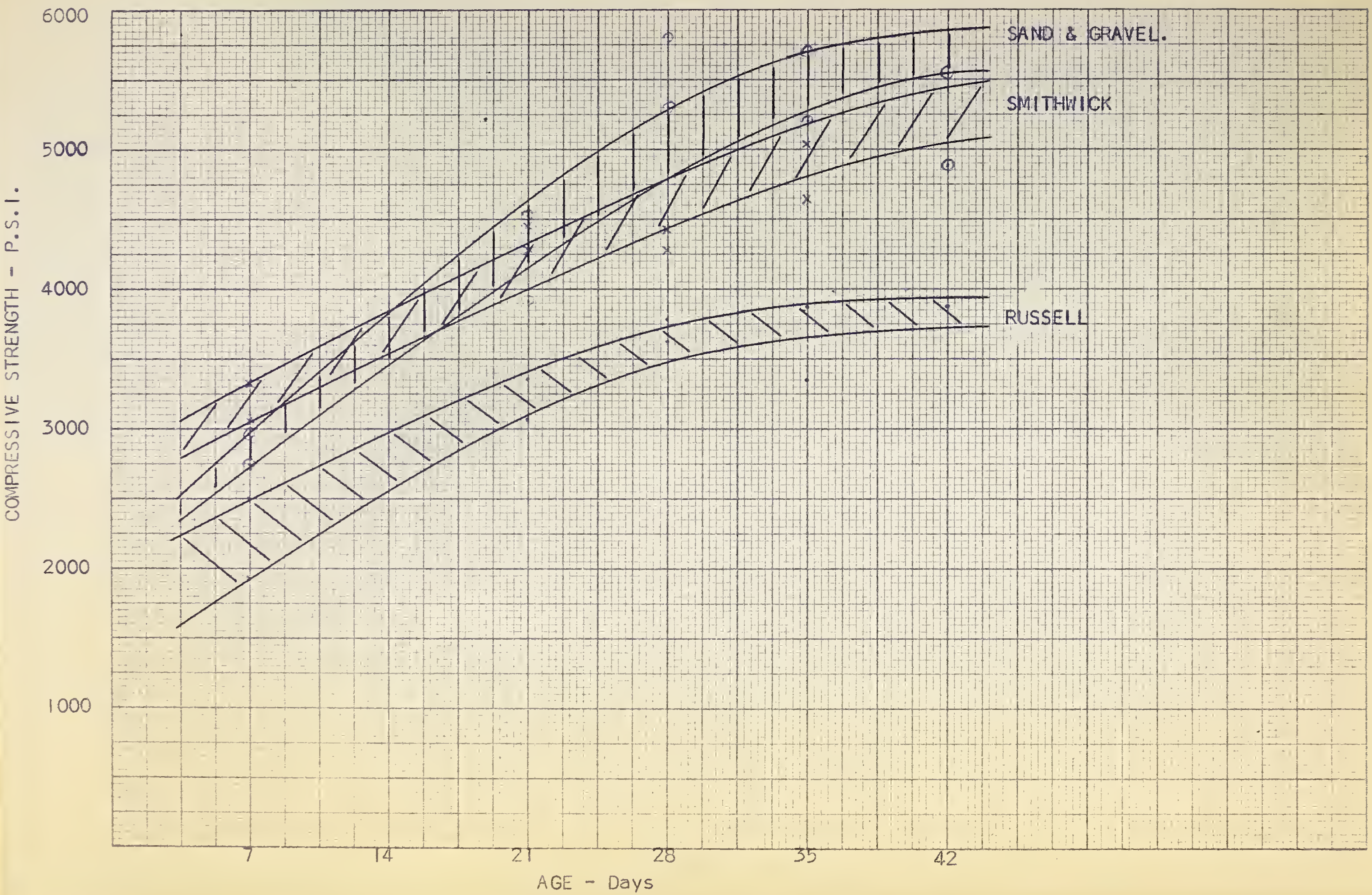


FIG. 2

AGE - STRENGTH CURVE - W/C = 0.50 ORDINARY MIXES

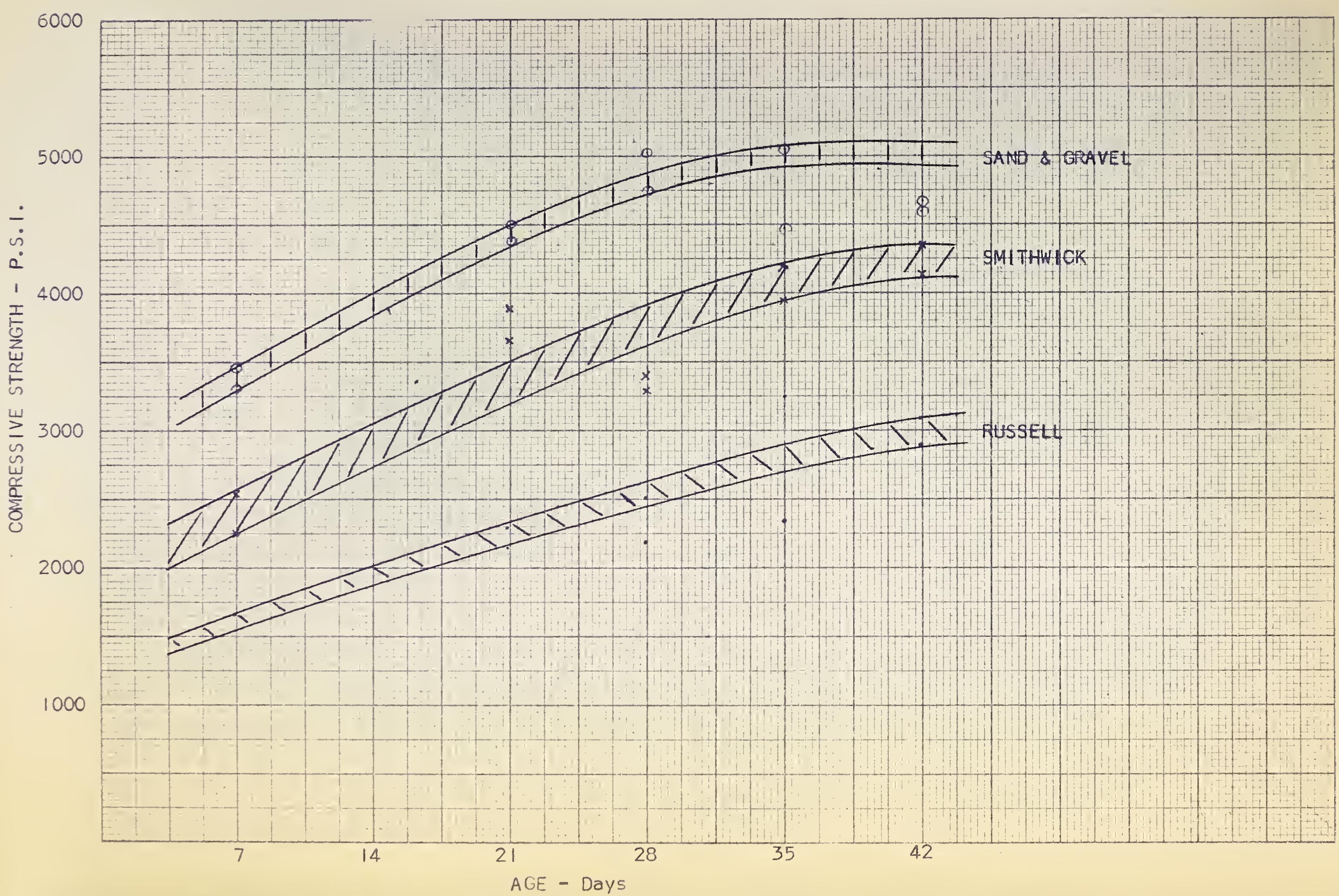


FIG. 3

AGE - STRENGTH CURVE - W/C = 0.60 ORDINARY MIXES

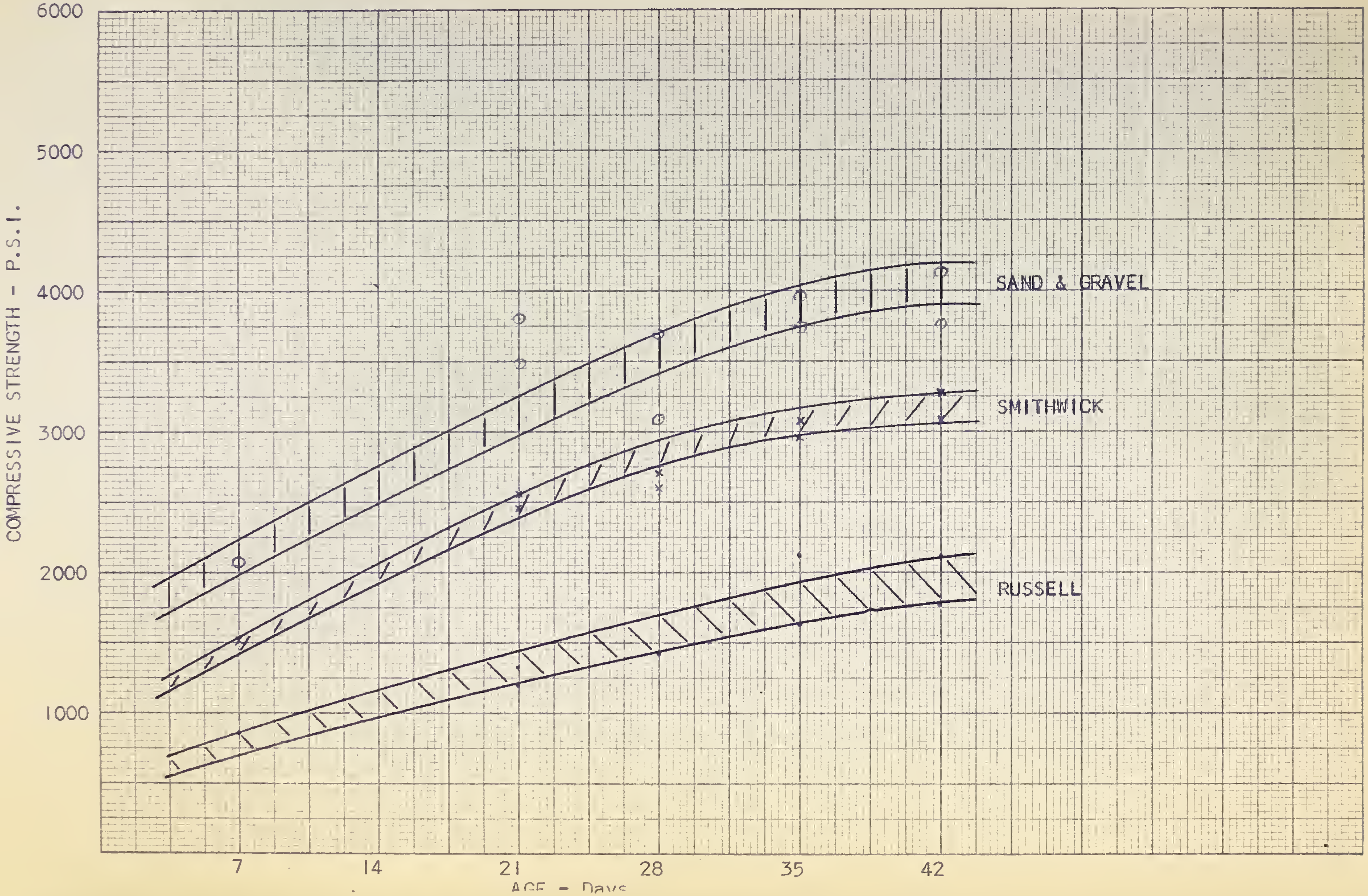


FIG. 4

AGE - STRENGTH CURVE - W/C = 0.70   ORDINARY MIXES

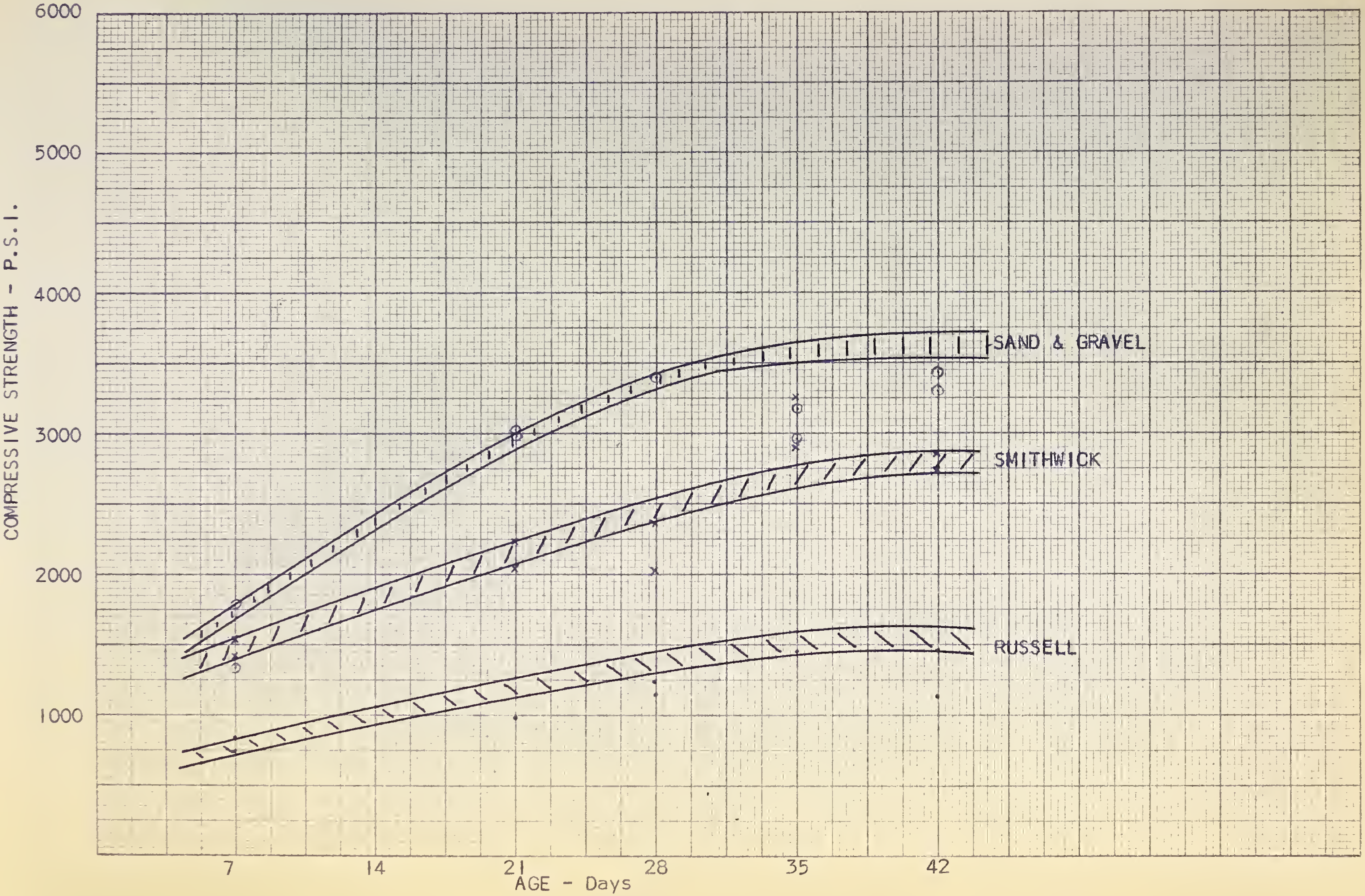


FIG. 5

COMPRESSIVE STRENGTH vs W/C RATIO ORDINARY MIXES

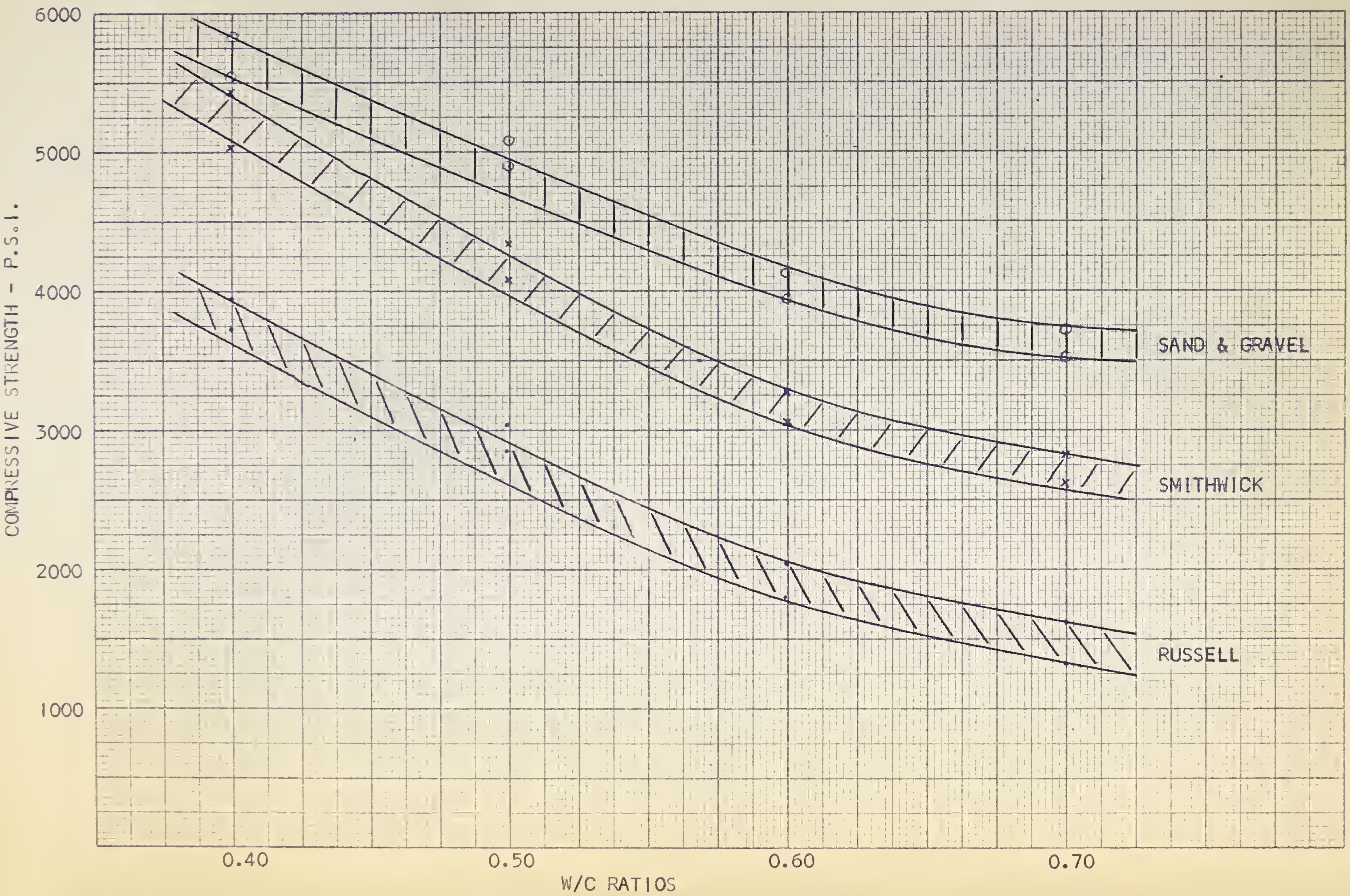


FIG. 6

AGE - STRENGTH CURVE - W/C = 0.40 A.E.A.

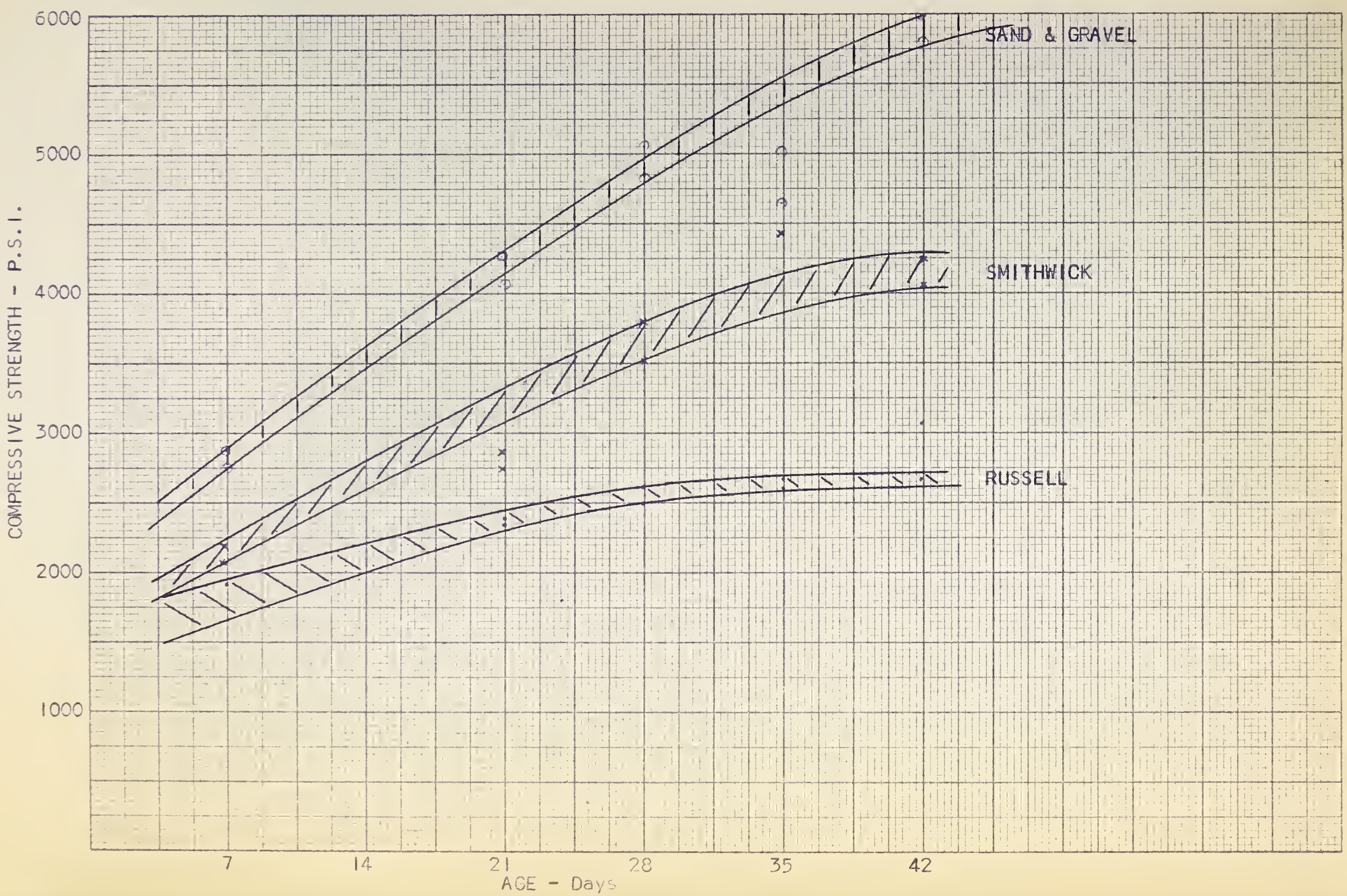


FIG. 7

AGE - STRENGTH CURVE - W/C = 0.50 A.E.A.

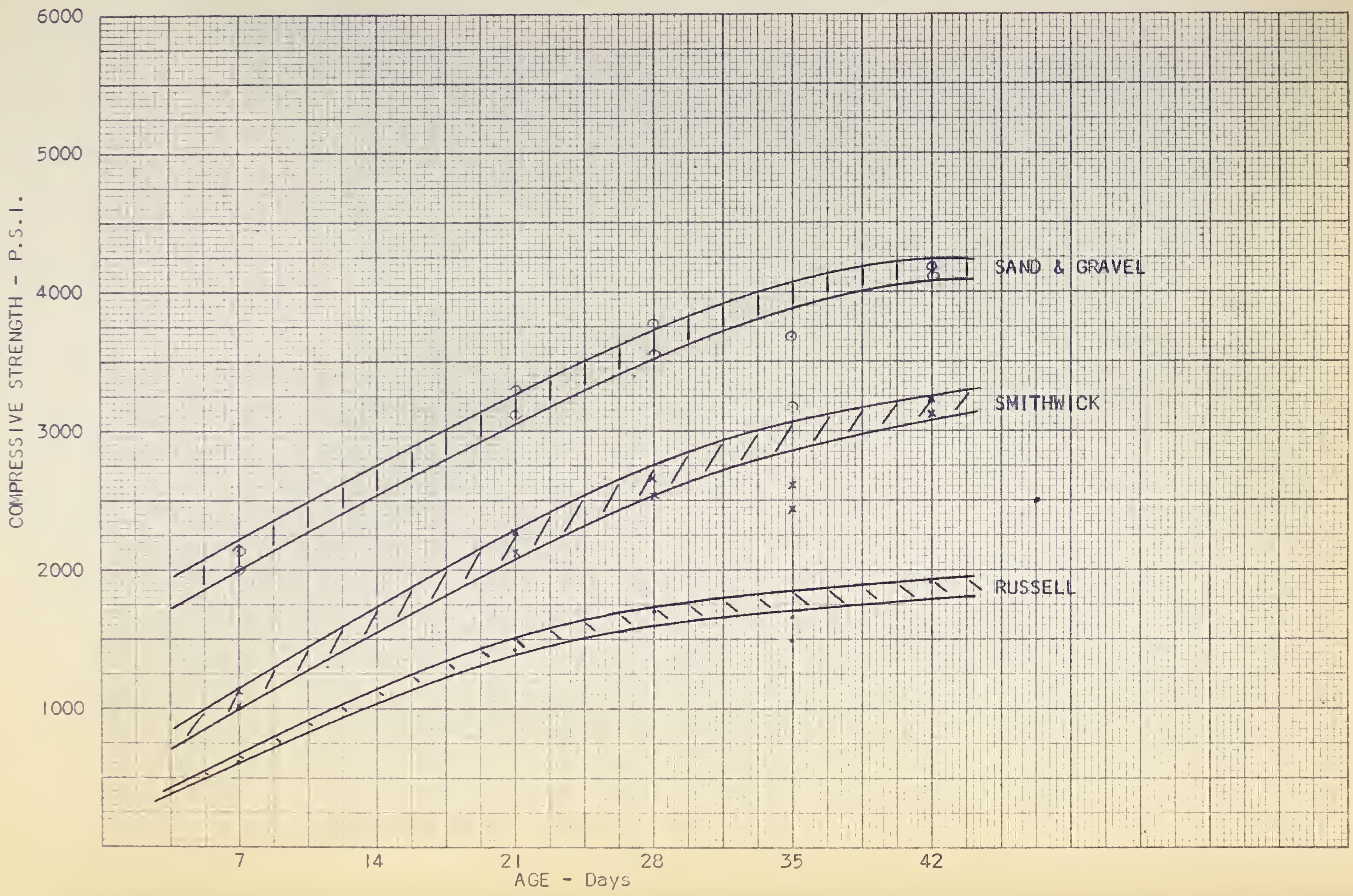


FIG. 8

AGE - STRENGTH CURVE - W/C = 0.60 A.E.A.

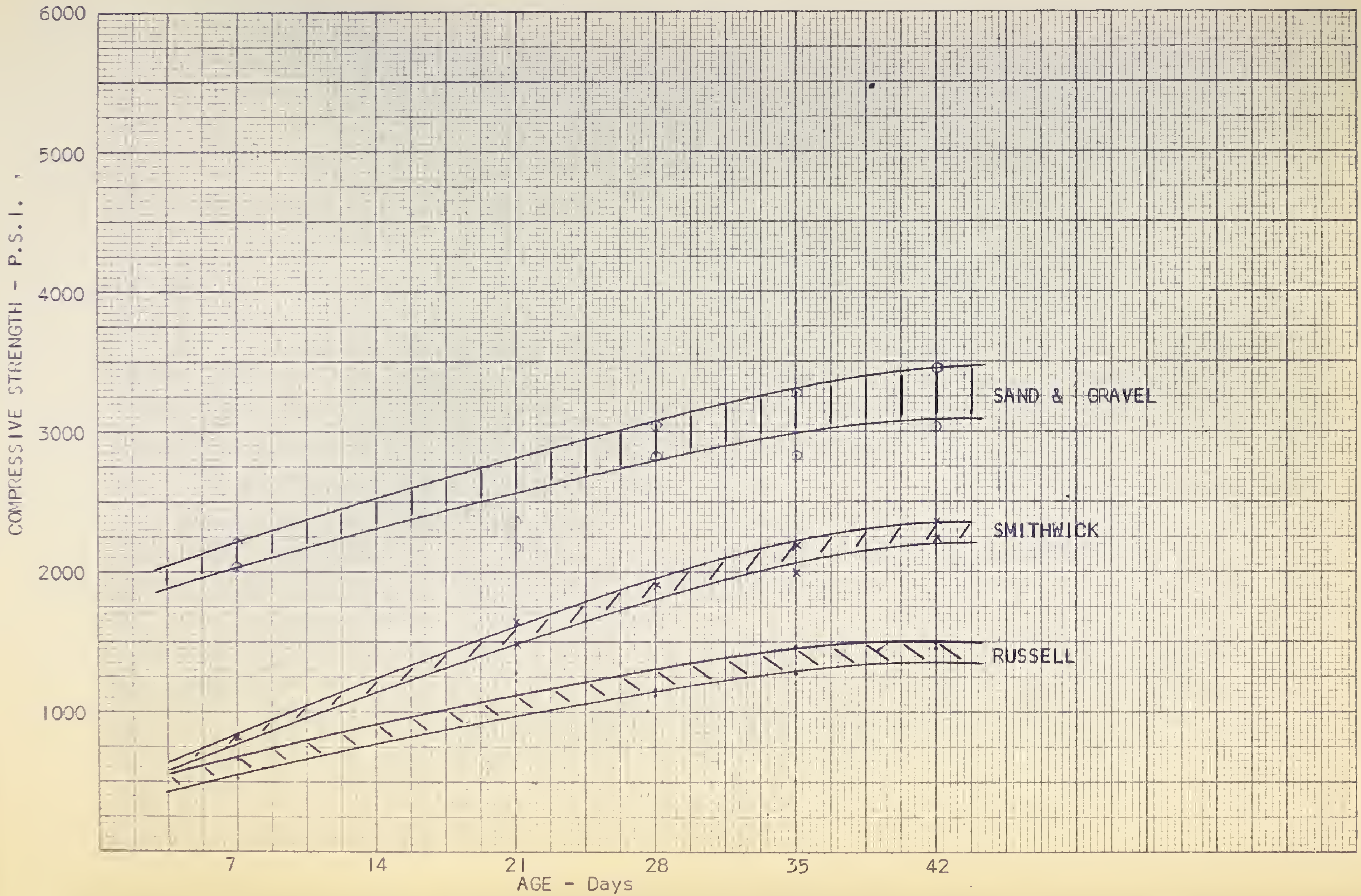


FIG. 9

AGE - STRENGTH CURVE - W/C = 0.70 A.E.A.

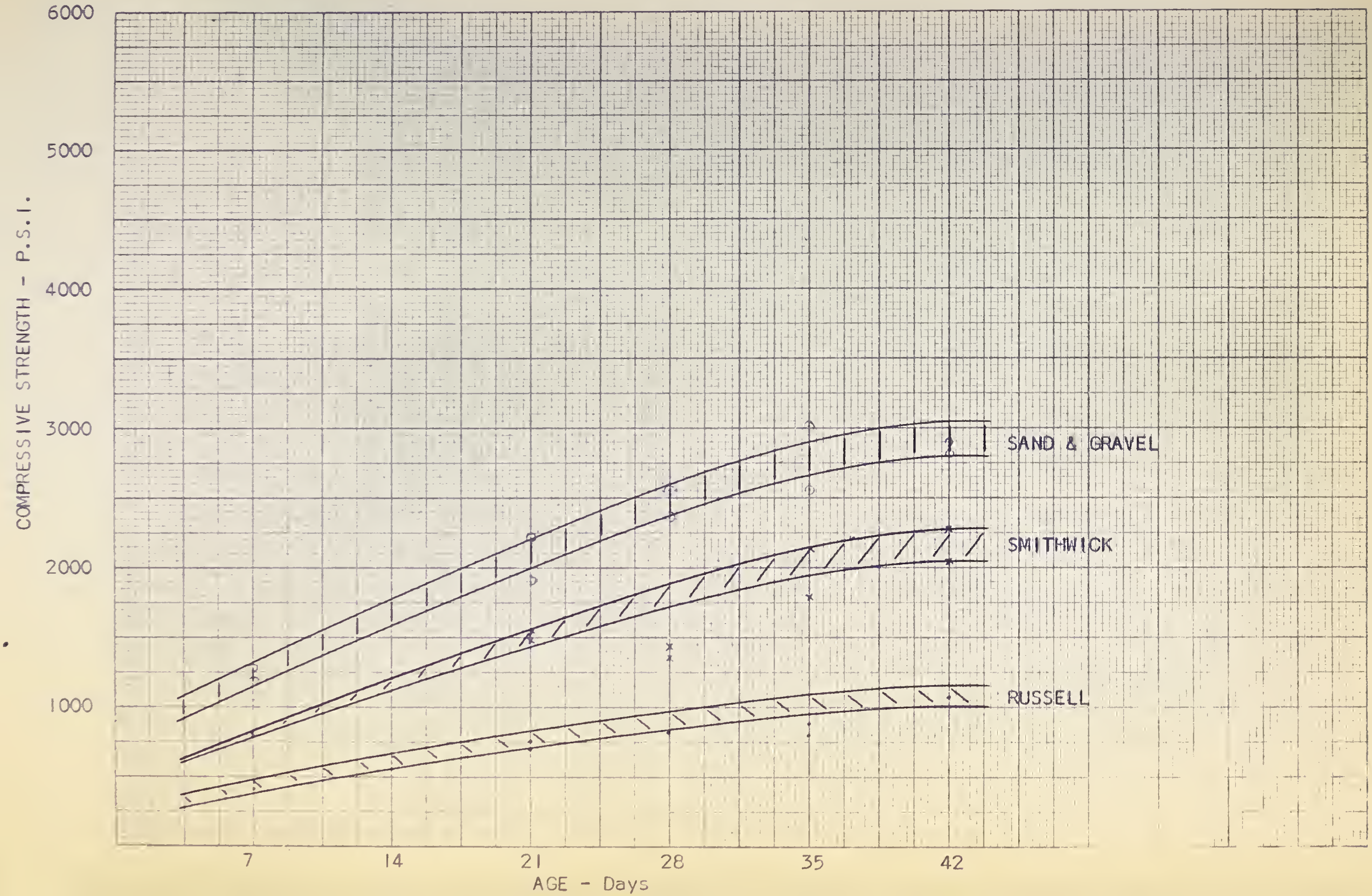


FIG. 10

COMPRESSIVE STRENGTH vs W/C RATIO A.E.A.

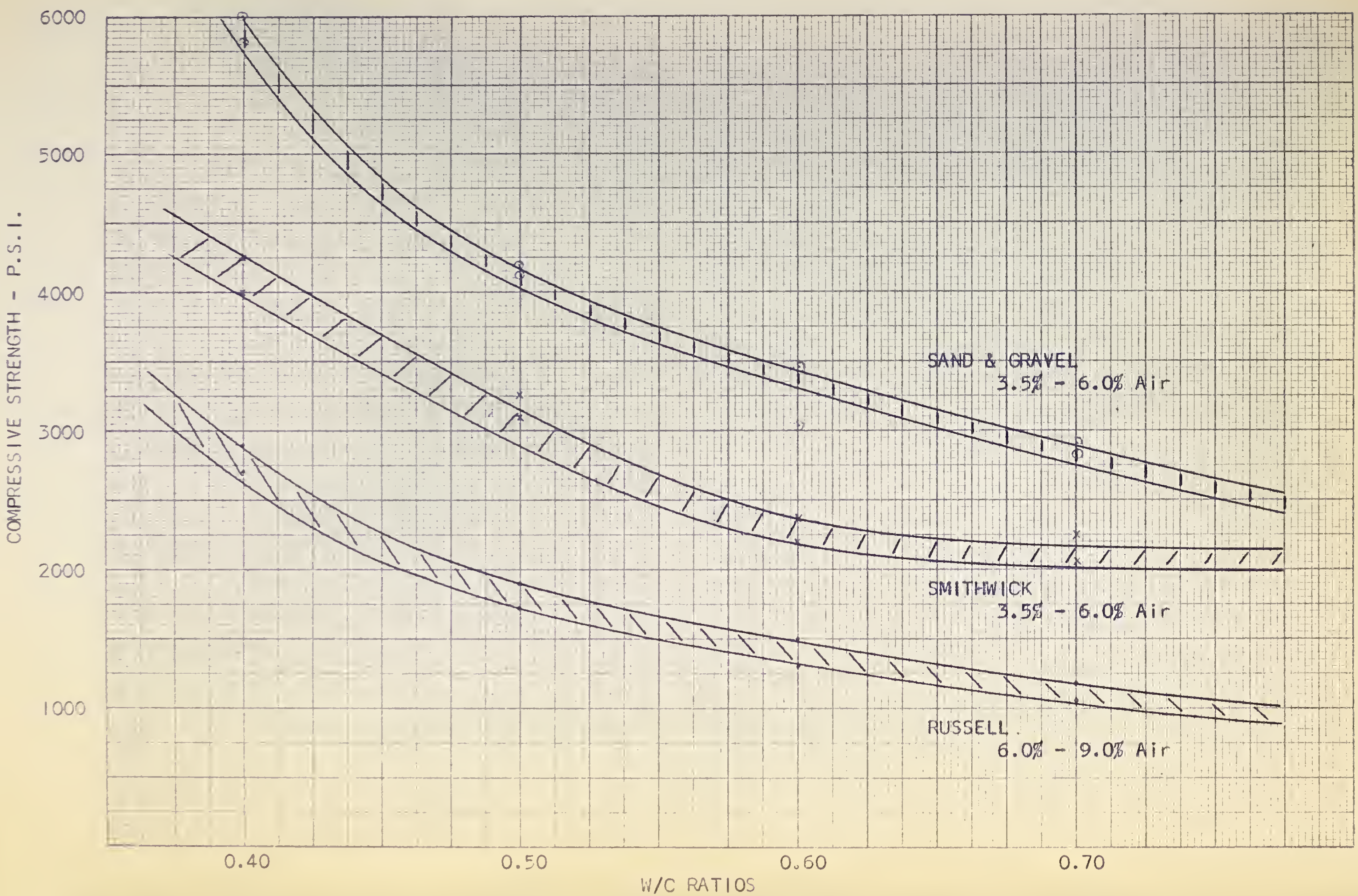


FIG. 11

COMPRESSIVE STRENGTH vs CEMENT CONTENT - SAND & GRAVEL

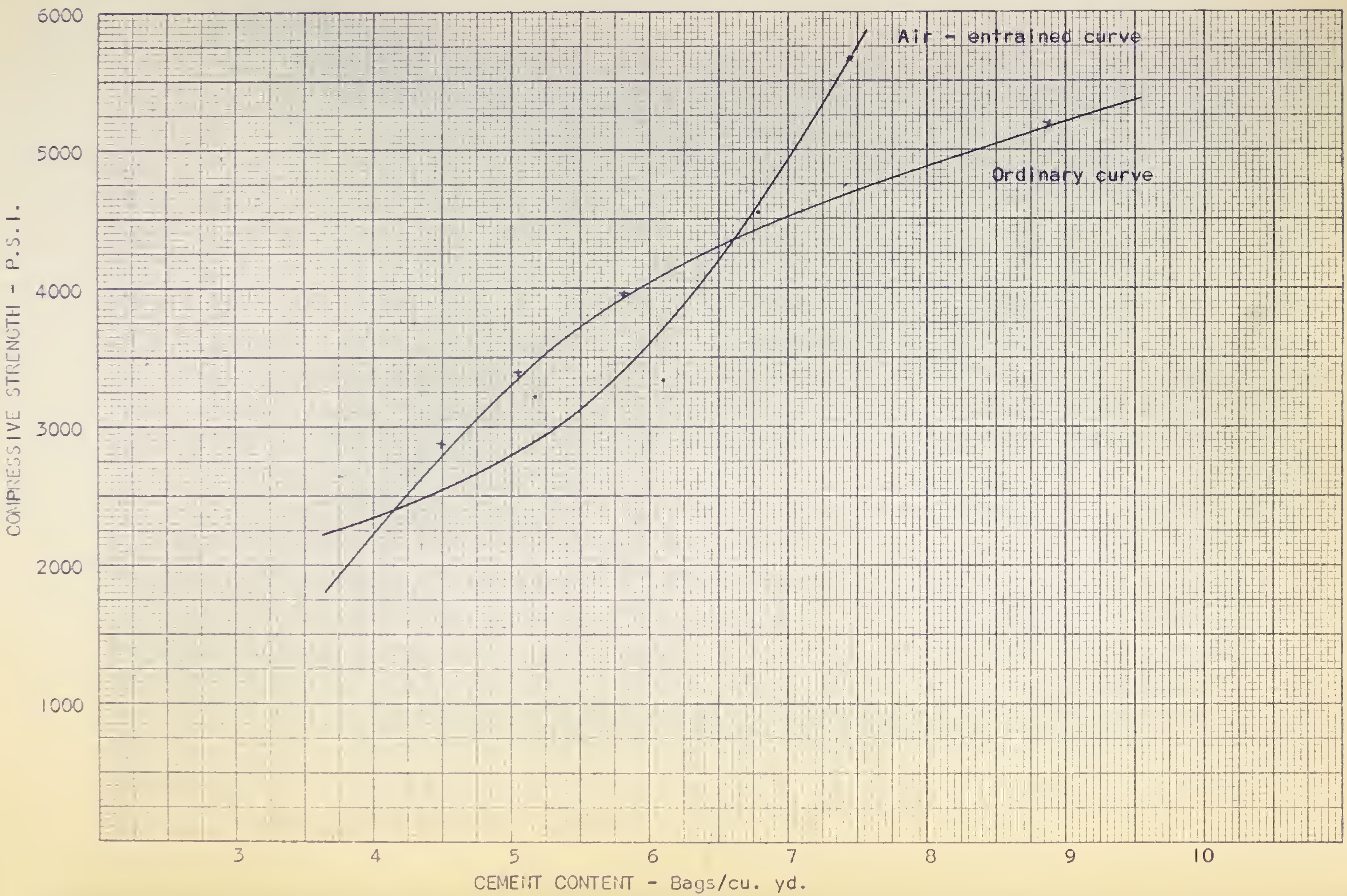


FIG. 12

COMPRESSIVE STRENGTH - CEMENT CONTENT - SMITHWICK'S AGG.

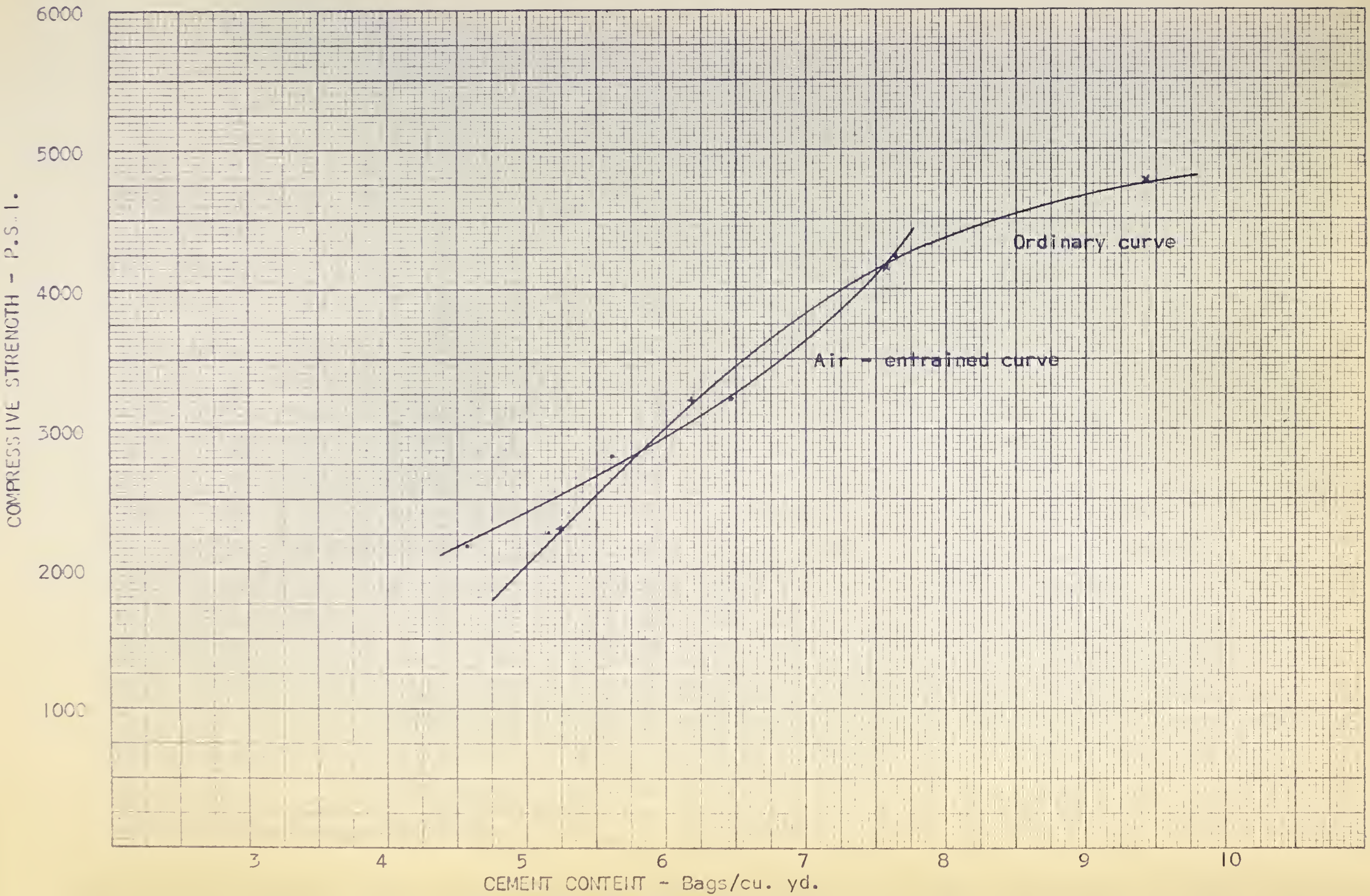


FIG. 13

COMPRESSIVE STRENGTH vs CEMENT CONTENT - RUSSELL'S AGG.

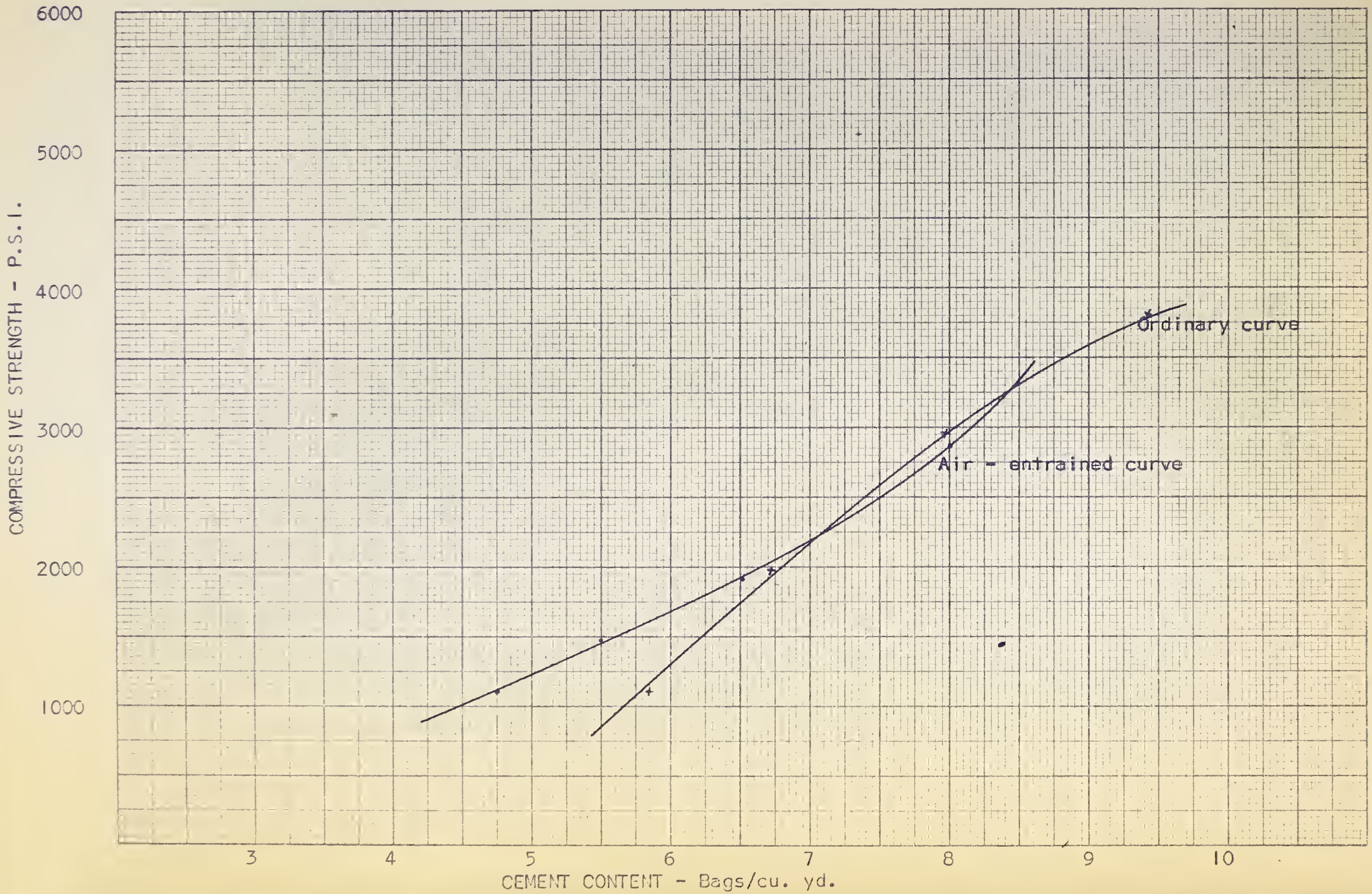
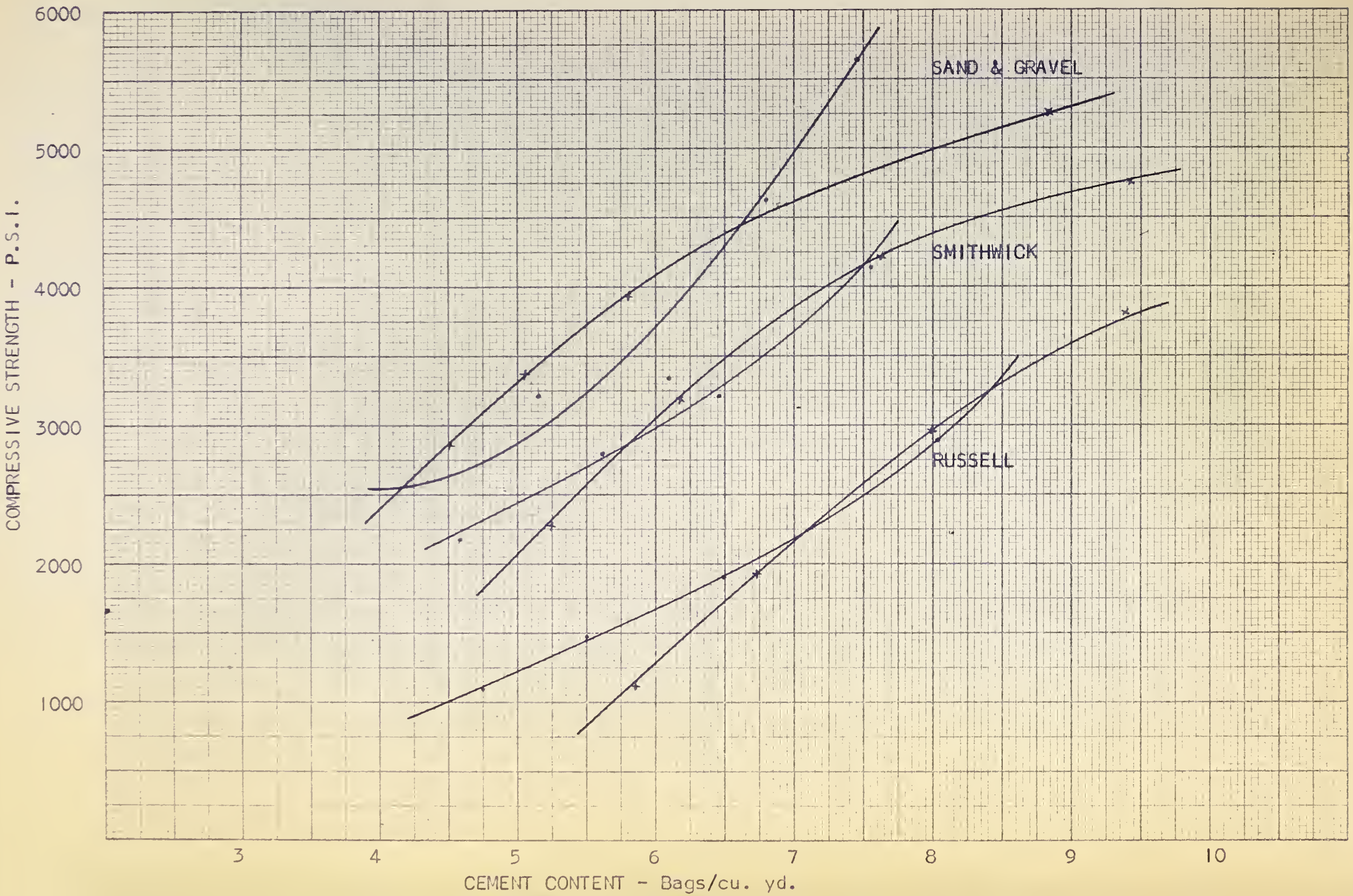


FIG. 14

COMPRESSIVE STRENGTH - CEMENT CONTENT



Chapter VI

Freezing and Thawing Tests

Testing Procedure

The $4\frac{1}{2}$" x $3\frac{1}{2}$" x 16" beams used for freeze-thaw tests were made up at the same time and of the same mixes as were used for the strength tests. The original intent was to put the beams in for test at an age of 28 days. This became impossible when it was apparent that the freeze-thaw apparatus would not be ready by this time. This was due to a time delay needed to increase the capacity of the hot and cold liquid tanks of the apparatus and its installation in the new Engineering Building. The first beams were put in at an age of 72 days. The ages at which the other beams were put in the apparatus are noted in Tables 11 and 12.

The procedure throughout the test period consisted of initial frequency readings prior to the test runs and then frequency readings at the end of 1, 2, 4, 8 and 12 cycles and then at approximately 11 or 12 cycle increments thereafter. In the case of the light weight beams, the 1, 2, 4 and 8 cycle readings were not necessary as there were no large decreases in frequencies initially as in the case of the ordinary sand and gravel concrete beams. This procedure was carried out, however, to ensure maximum reliability of results.

Photographs were taken to show the effects of the rigorous cycles on the beams. These were taken upon completion of the 300 cycles or more and also at intervals throughout the tests to indicate the beam break-down as it occurred. This was especially essential in the case of the ordinary beams made with sand and gravel aggregate which broke down prior to completion of

300 cycles.

To ensure that all cycles were complete and that proper temperature ranges were maintained, a Brown temperature recorder was employed.

Two durability factors are listed in the tables. One durability factor was based on the sonic modulus of the concrete while the other is based on the modulus of rupture.

The durability factors are computed in the following way:

$$D.F.E. = \frac{P\ N}{M}$$

D.F.E. = durability factor in % of dynamic E at zero cycles.

P = Relative E of 50%, or greater, at time of completion of tests, based on the E at zero cycles.

N = number of cycles at which P reaches 50% or the ultimate number of cycles of the test.

M = ultimate number of cycles of the test. In this case 300 cycles was a complete test.

$$D.F.R. = \frac{R_2}{R_1}$$

D.F.R. = durability factor in %, ratio of modulus of rupture of freeze-thaw beam to control beam.

R_2 = modulus of rupture of freeze-thaw beam upon completion of tests.

R_1 = modulus of rupture of control beam.

The modulus of rupture was computed using the formula

$$R = \frac{M.\ y}{I}$$

where

$R =$ modulus of rupture in p.s.i.

$M =$ bending moment (maximum) required to break beam in inch-lbs.

$I =$ moment of inertia about neutral axis

$= \frac{b.d^3}{12}$ inches^4 for rectangular figure

$y = \frac{1}{2}$ depth of specimen in inches

Three durability factors are normally considered in connection with concrete. They are

(1) Durability factor based on the sonic modulus of elasticity.

(2) Durability factor based on the modulus of rupture.

(3) Durability factor based on loss of weight of specimen.

Of the three durability factors, the one based on the sonic modulus of elasticity is considered the more indicative of actual conditions. The deterioration of the beam is readily measured by the use of sonic equipment which yields its natural frequency. As the beam deteriorates the natural frequency is reduced and consequently the elastic modulus. This enables plotting of results with the test in progress and actual determination of the number of cycles required to cause failure of the specimen.

The durability factor, with respect to the modulus of rupture, gives a result based on an initial and final reading on the beam. The last mentioned method which is dependent upon loss in weight was not used.

Test Results

The complete results of the freezing and thawing tests are shown in Tables 11 and 12. The results are presented graphically in Figures 15 to 38. The results are conclusive and show good correlation. The ordinary mixes of

the sand and gravel show a zero durability factor. The graphs (Figures 15 to 18) show the modulus of elasticity falling below 50% considerably earlier than the 300 cycles which constituted a complete test. In contrast, the light weight concrete in the ordinary mixes shows very acceptable durability factors. The durability factors with regard to elastic modulus varied from a high of 94 to a low of 64. The low durability factor showed up, of course, in the low strength range.

The results of the air-entrained mixes (Table 12) show good durability factors for all mixes - both sand and gravel and light weight. This is as expected as the purpose of air-entrainment is one or both of two factors - increased workability and increased durability. There is little to choose between the various mixes. All show durability factors in the same range. The sand and gravel mixes benefited the most due to the air-entrainment; next the Smithwick aggregate and lastly Russell's aggregate which showed excellent results for both tests. On a strength basis, Russell's aggregate was superior in durability quality.

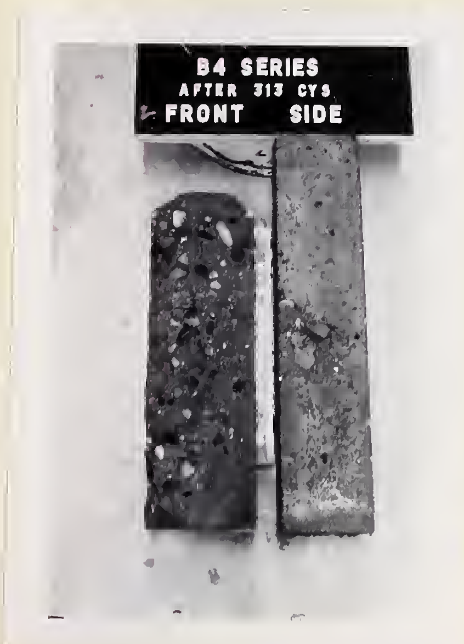
In these tests some beams were left in the freeze-thaw apparatus far in excess of the required 300 cycles. The 0.5 w/c ratio mix of the sand and gravel series, was left in for 542 cycles and showed no adverse effects. It still had a durability factor with respect to elastic modulus of 94.3%. The 0.4 w/c ratio mix of Russell's aggregate was left in for 403 cycles and showed an identical durability number of 94.3%.

The ages at which the beams were put in the freeze-thaw apparatus vary considerably, ranging from a minimum of 61 days to a maximum of 106 days. This, as mentioned before, was an unavoidable circumstance. However, since the period of curing was in excess of 60 days it was felt that the overall effect

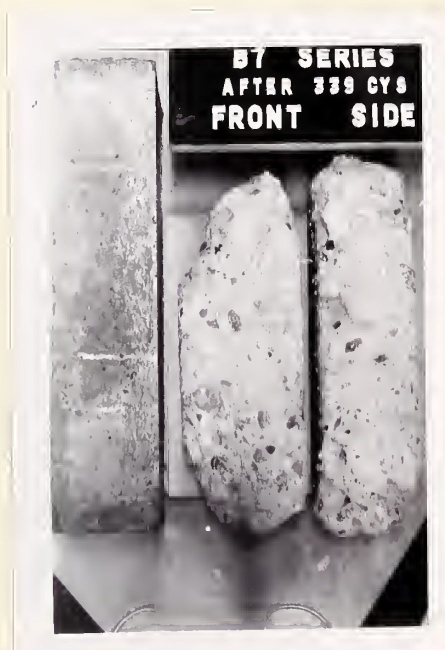
of increased curing time would not be very great and since it was impossible to evaluate any increase in durability due to age of curing it was neglected.

Photograph No. 2 – Typical Beam Specimens before being put into Freeze–Thaw.

The photographs taken of the beams during and after freeze–thaw tests are self explanatory. The number of cycles at which the photographs were taken are shown clearly in the photographs as well as data relative to the various mixes. The extent of breakdown of the ordinary sand and gravel mixes is clearly shown in contrast to the light weight beams which are in relatively good shape. Photographs of the air–entrained mixes were taken only at the completion of test runs since there was little or no visible breakdown.

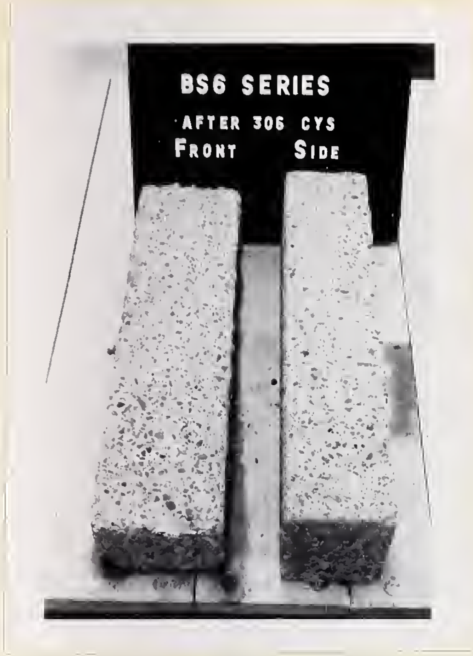


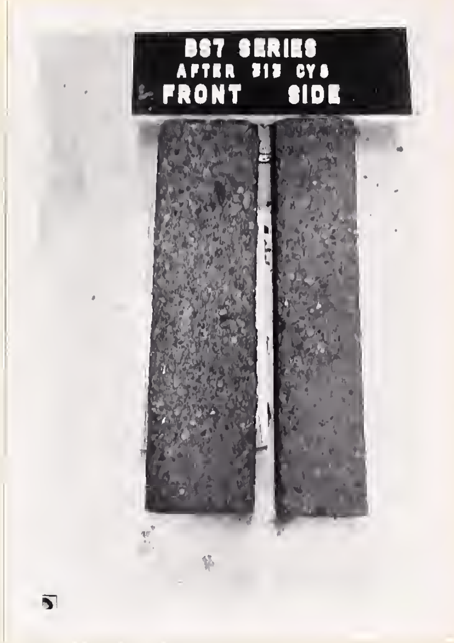
Photographs Nos. 3-6 - Beams B4 and B5 during and after completion of Freeze-Thaw tests.

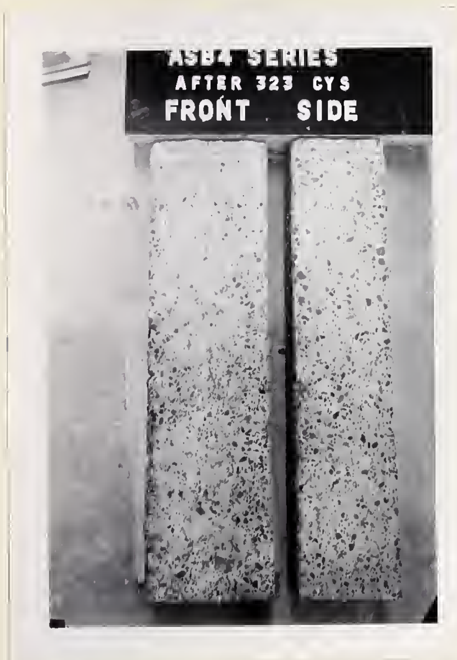


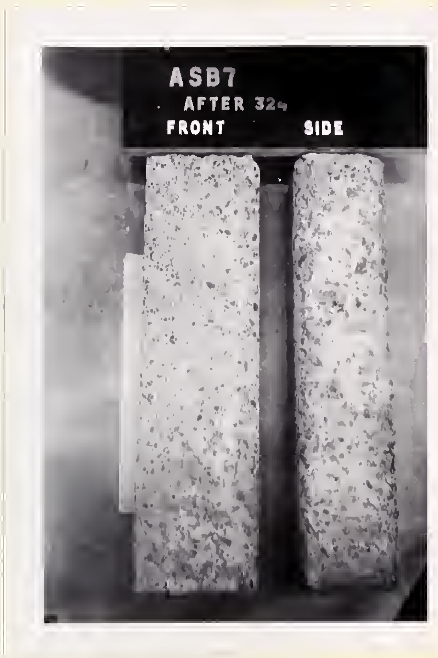
Photographs Nos. 7–10 – Beams B6 and B7 during and after completion of Freeze–Thaw tests.

Photographs Nos. 11–14 – Beams AB4, AB7, ARB4 and ARB7 after completion of Freeze–Thaw tests.

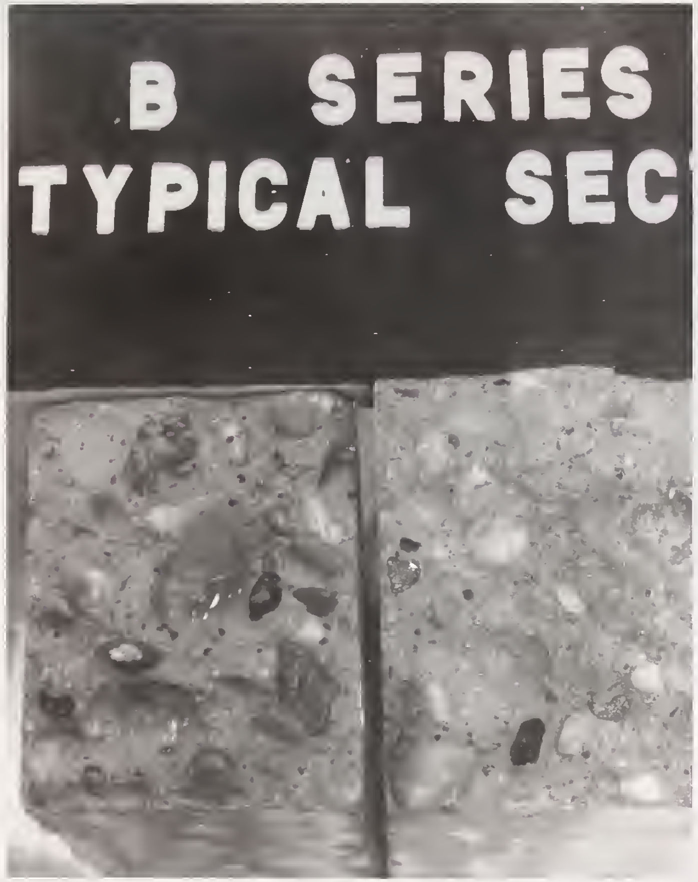


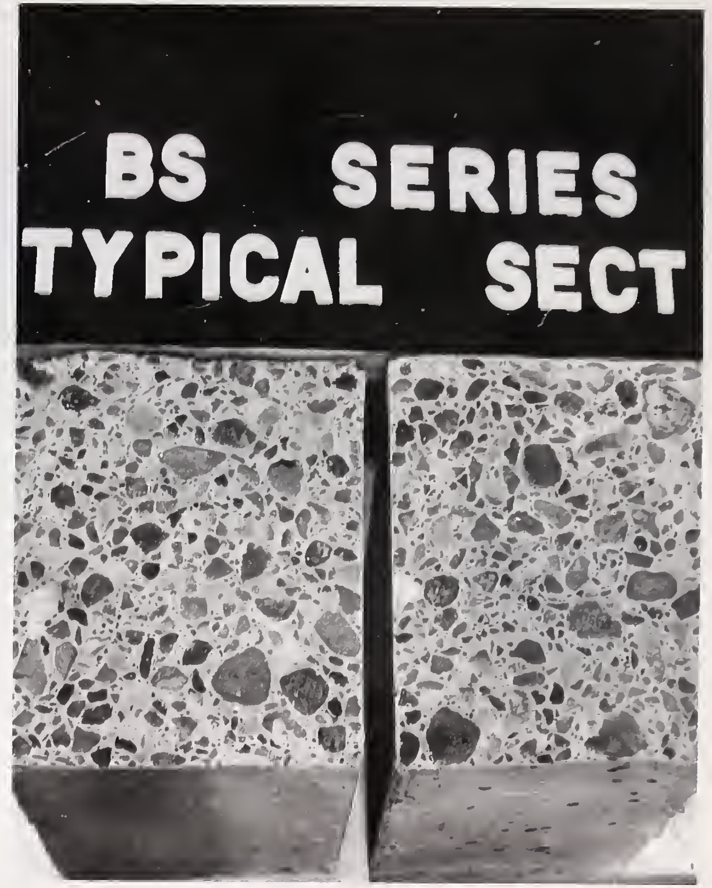






Photographs Nos. 15-18 - Beams BS6, BS7, ASB4 and ASB7 after completion of Freeze-Thaw tests.





Photographs Nos. 19-20 - Typical Sections of Sand & Gravel Concrete and Light Weight Concrete

Sections of the beams - Sand & gravel and light weight are shown. It shows the difference in texture and composition. See page 61 .

Discussion of Results

The test results seemed to bear out what had been assumed initially - that the light weight concrete would have superior durability qualities because of the porous nature of the aggregate. It was thought that this porous nature would allow for the expansion and the contraction caused by the alternating cycles of freezing and thawing. This porous nature was to absorb the expansion and contraction in contrast to the sand and gravel which could not do this unless minute bubbles of air were put into it for this purpose. The resulting tests seemed to bear out this statement as the light weight concrete had much superior durability to its sand and gravel counterpart where air-entrainment was not used. An inspection of the photograph showing the sections of the two types of concrete shows this. The light weight has what might be called built in air-entrainment due to its porous nature. Another consequence of this might be noted and also present the reason for the superiority of the expanded clay to the expanded shale. Although the two sections of light weight concrete are not shown, it was noted that the expanded clay (Russell's aggregate) seemed to have a coarser bloat, that is a relatively larger porous nature. This would give it the superior durability as previously pointed out.(21)

There is comparatively close agreement between the two durability factors as can be seen from the table. The same range of values is nearly always maintained. This holds true for both the ordinary and air-entrained mixes. It would seem to indicate that where sonic equipment is not feasible, results within an acceptable range could be expected from the use of the

durability factor with respect to the modulus of rupture. There are some deviations from this statement in the table but on the whole, as stated before, the correlation between the two factors is good when one considers the property being measured.

The tests bring out one basic conclusion – that is, that light weight concrete made using an expanded aggregate has durability far superior to ordinary sand and gravel concrete. The entraining of air in sand and gravel concrete puts it on a par with that of air-entrained light weight concrete.

In connection with the study of durability of light weight concrete, it might be added that information on this subject is by no means plentiful. Kluge, Sparks and Tuma [(4)] in their article entitled "Light Weight Concrete" give some values for durability but other than that, little can be found.

Also of interest in connection with durability is the reported resistance of light weight concrete to sea water. The recently refloated U.S.S. Selma, after being submerged in sea water for 34 years, showed no signs of deterioration. Reinforcing steel with a 5/8" cover of concrete showed no harmful effects from the submergence.

In concluding this chapter some note might be made of the resistance to freezing and thawing due to compressive strength of the concrete. This is clearly shown in the light weight concrete made with Russell's aggregate with no air-entrainment. The 0.4 w/c shows a durability factor of 93.5 gradually falling off to 64.0 for the 0.7 w/c ratio. This was also shown in the sand and gravel mixes of this same series. The 0.4 w/c ratio beams were able to survive for 228 cycles whereas those of 0.7 w/c reached a zero durability factor at 143 cycles. Strength then, becomes a relatively important characteristic when dealing with durability of concretes and where an extreme condition

of weathering could be encountered, air entrainment and strength would have to be important factors in the design.

Results of Freeze-Thaw Tests on Ordinary Mixes

Beam No.	Compressive Strength p.s.i. 42 day cyl.	Initial E Values p.s.i.x10^6	Initial R Values p.s.i.	No. of Freeze-Thaw cycles to test	Age at time of test – Days	D.F.E. at end of test	D.F.R. at end of test
B4	5200	4.62	1013	228	92	0.0	22.1
B5	4620	4.79	920	204	93	0.0	21.6
B6	3950	3.86	865	180	94	0.0	8.1
B7	3370	3.82	658	143	106	0.0	0.0
BS4	5210	2.11	1163	323	74	90.5	88.5
BS5	4215	1.80	995	315	82	86.5	100.0
BS6	3220	1.76	728	306	83	76.2	88.8
BS7	2805	1.65	808	313	95	82.2	90.0
BR4	3795	1.83	1195	320	72	93.5	83.0
BR5	2975	1.79	905	323	73	87.4	99.2
BR6	1940	1.63	808	306	76	86.2	98.9
BR7	1140	1.40	620	304	73	64.0	67.4

TABLE 12

Results of Freeze–Thaw Tests on Air–Entrained Mixes

Beam No.	Compressive Strength p.s.i. 42 day cyl.	Initial E Values p.s.i.x10^6	Initial R Values p.s.i.	No. of Freeze–Thaw cycles to test	Age at time of test – Days	D.F.E. at end of test	D.F.R. at end of test
AB4	5905	4.23	1154	330	100	96.6	98.4
AB5	4150	4.28	787	542	61	94.3	100.0
AB6	3230	4.40	810	327	87	94.5	100.0
AB7	2865	3.94	654	327	75	97.1	100.0
ASB4	4155	1.99	1045	323	79	91.6	82.3
ASB5	3190	1.68	772	320	83	95.2	99.7
ASB6	2290	1.57	923	320	82	94.5	74.6
ASB7	2170	1.44	816	324	82	92.6	99.9
ARB4	2890	1.58	730	403	94	94.3	71.2
ARB5	1915	1.36	639	345	95	94.6	100.0
ARB6	1475	1.29	616	345	94	92.3	100.0
ARB7	1110	1.06	453	330	104	92.9	100.0

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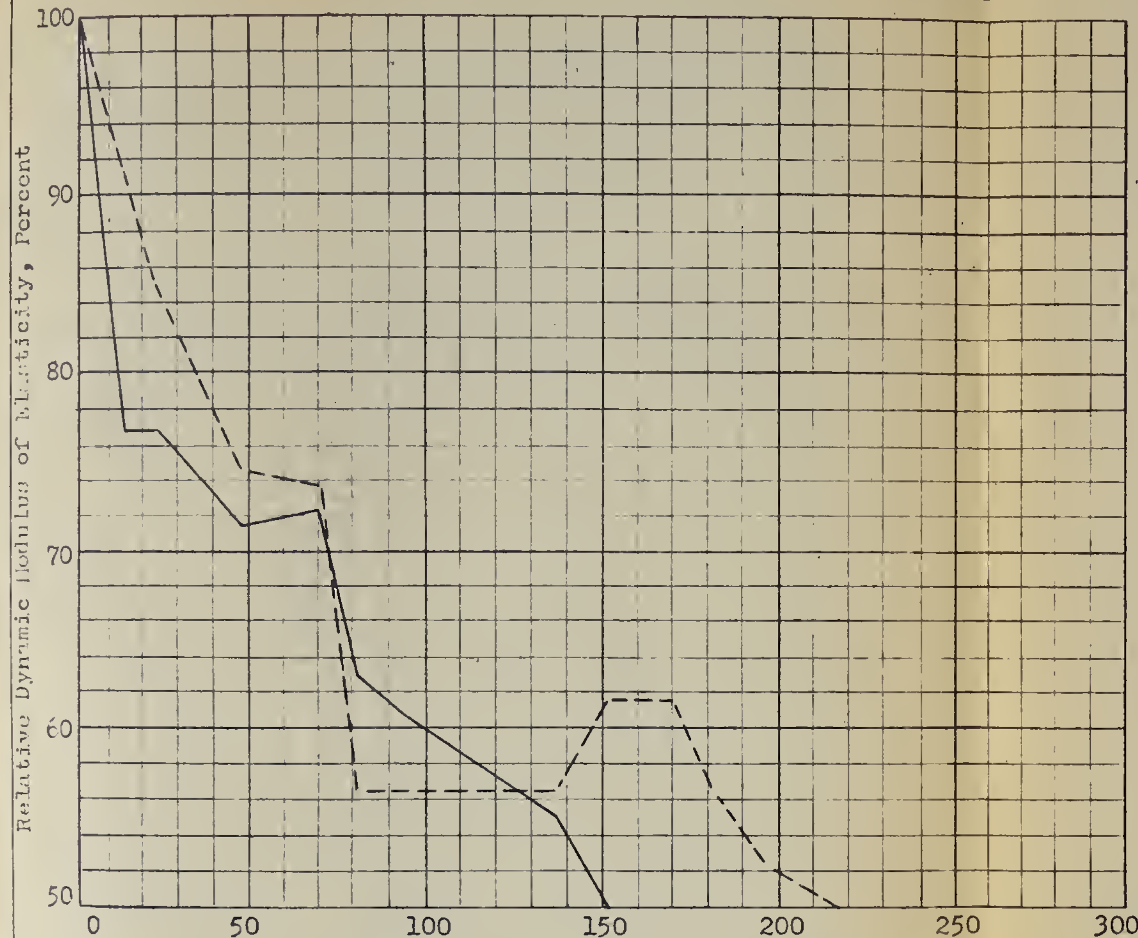
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg.	
Elk Island Sand 1270#	Mix No. 1
	Cement Content ......775#
	Water Content ......310#
Coarse Agg.	Air Content 1.5%
Horst Hills 1715#	W/C 0.40
	V/C ....----
Cement	Slump ....3"
Portland Cement	Age at Test ....92 days
	Avg. Comp. Strength..5200 p.s.i.
	Avg. Flex. Strength..1013 p.s.i.
	Avg. Density ....---- 149.5#/cu.ft.
	Avg. Flex. Strength after test....---- 224 p.s.i.

Beam No.	Symbol	DFE at 300 cycles	No. Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
B4-1B	———	0.0	151	22.1	-------
B4-1C	— —	0.0	218	22.1	-------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
B4-1B	Standard	25° - 70°F	Beam very badly spalled; after 228 cycles unable to read.
B4-1C	Standard	25° - 70°F	Same as above.

Project: FIG. 15

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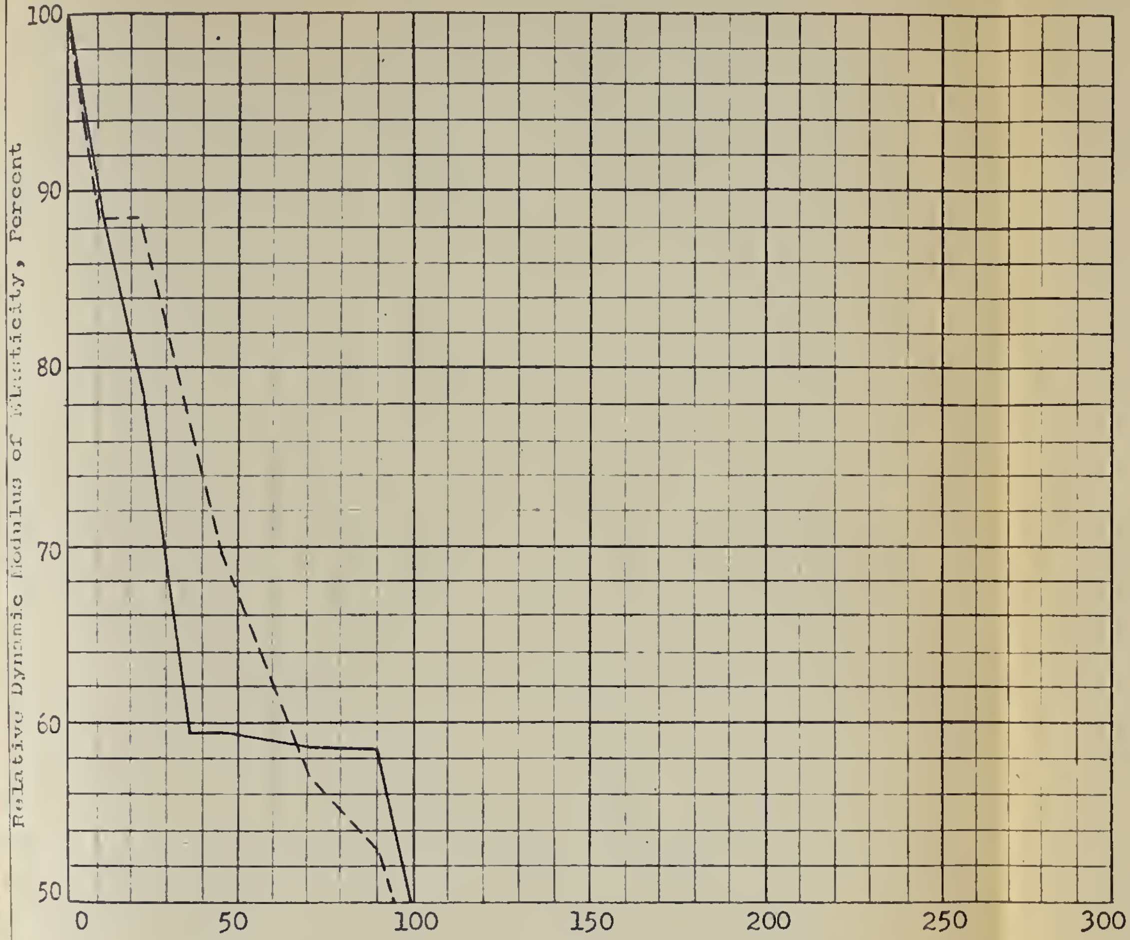
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg. Elk Island Sand 1387#	Mix No. ....2 Cement Content ......620# Water Content ....310 Air Content ....2.0% W/C ....0.50 V/C ....—— Slump ....3" Age at Test ....93 days Avg. Comp. Strength..4620 p.s.i. Avg. Flex. Strength..920 p.s.i. Avg. Density ....—— 146.3#/cu.ft. Avg. Flex. Strength . after test.....—— 199 p.s.i.
Coarse Agg. Horse Hills 1728#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No. Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
B5-1A	———	0.0	99	21.6	———
B5-1B	— —	0.0	95	21.6	———

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
B5-1A	Standard	25°-70°F	Beams very badly spalled;
B5-1B	Standard	25°-70°F	Same as above.

Project:

FIG. 16

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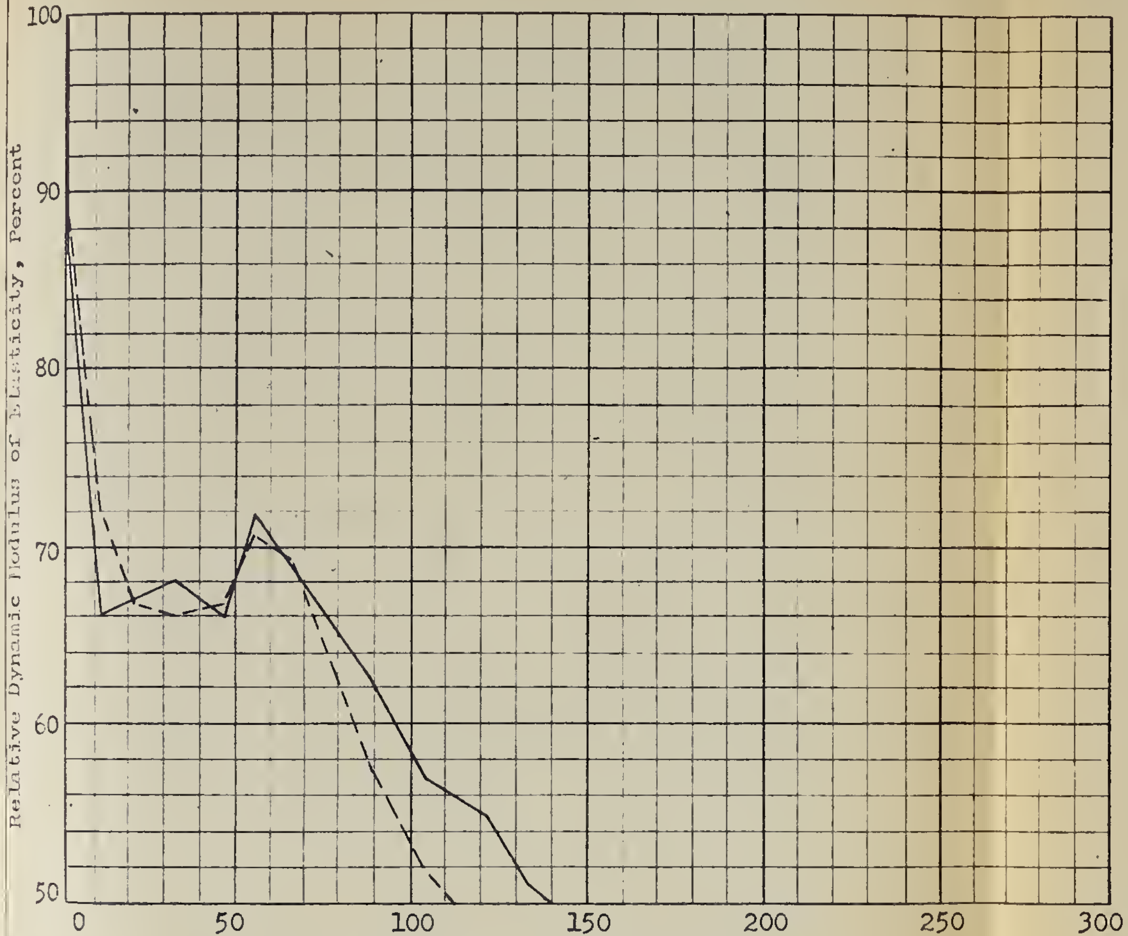
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg. Elk Island Sand 1492#	Mix No. .... 3 Cement Content ...... 517# Water Content .... 310# Air Content .... 2.0% W/C .... 0.60 V/C .... --- Slump .... 3" Age at Test .... 94 days Avg. Comp. Strength.. 3950 p.s.i. Avg. Flex. Strength.. 865 p.s.i. Avg. Density .... --- 147.0#/cu.ft. Avg. Flex. Strength after test..... ---70 p.s.i.
Coarse Agg. Horse Hills 1712#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No. Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
B6-1A	———	0.0	140	8.1	———
B6-1C	— —	0.0	112	8.1	———

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
B6-1A	Standard	25°-70°F	Beams very badly spalled.
B6-1C	Standard	25°-70°F	Same as above.

Project:

FIG. 17

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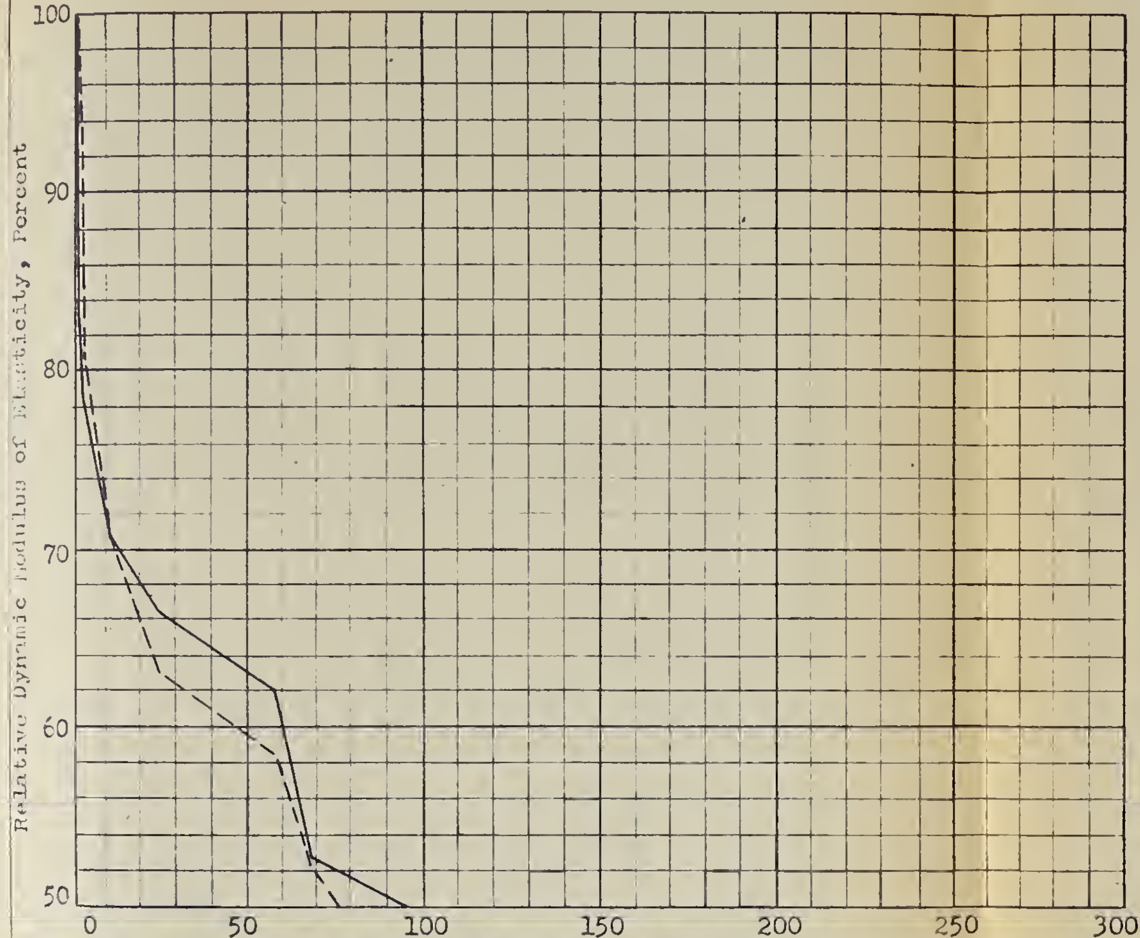
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg. Elk Island Sand 1586#	Mix No. ....4 Cement Content ......443# Water Content ....310# Air Content ....2.2% W/C ....0.70 V/C ....---- Slump ....3" Age at Test ....106 days Avg. Comp. Strength..3370 p.s.i. Avg. Flex. Strength..658 p.s.i. Avg. Density ....---- 145.6#/cu.ft. Avg. Flex. Strength after test.....----0.0 p.s.i.
Coarse Agg. Horse Hills 1635#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.P = 50%
B7-1B	———	0.0	95	0.0	--------
B7-1C	— —	0.0	76	0.0	--------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
B7-1B	Standard	25°-70°F	Sides & edges very badly spalled. Unable to read after 143 cycles.
B7-1C	Standard	25°-70°F	Same as above.

Project:

FIG. 18

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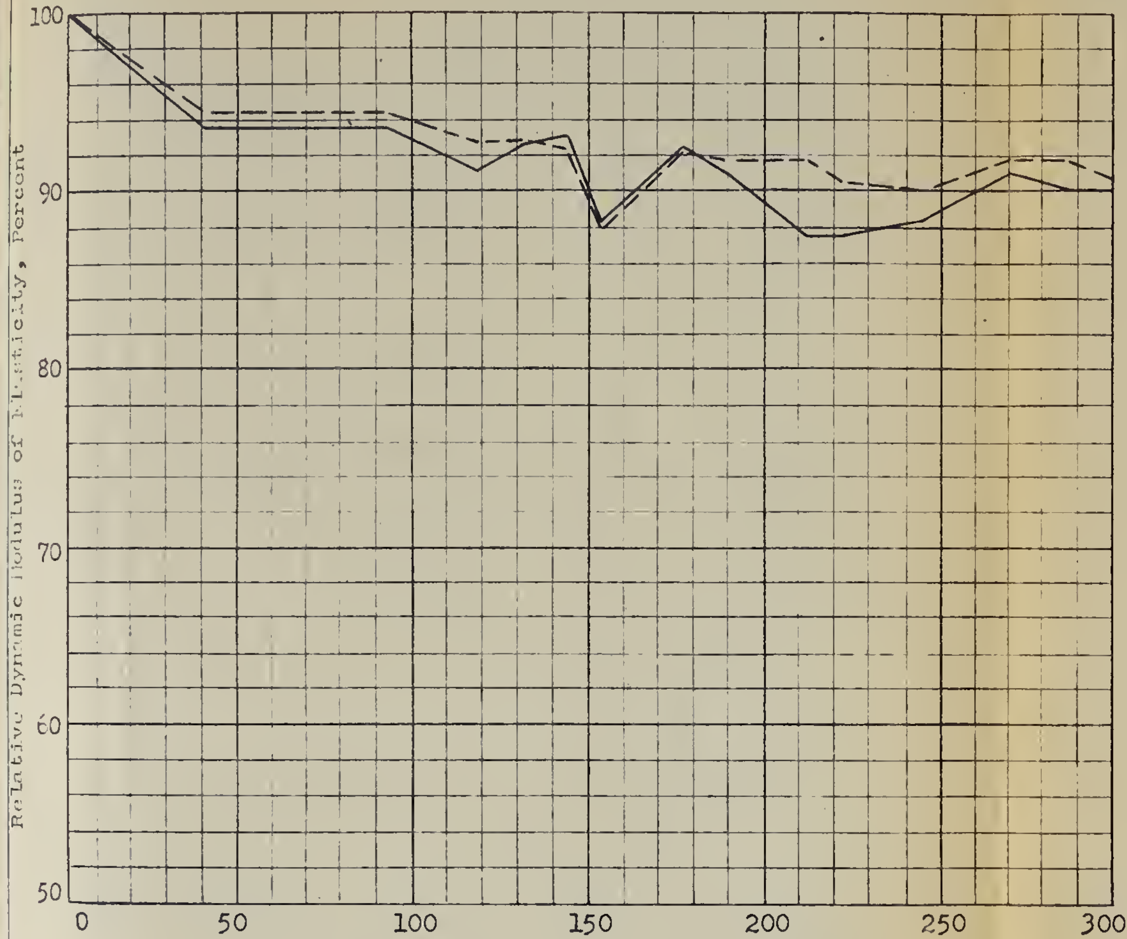
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg. Smithwick's Fines - 1120# Intermediate - 300#	Mix No. 1S
	Cement Content ...... 825#
	Water Content .... 330#
	Air Content 5.0%
Coarse Agg. Smithwick's 481#	W/C 0.40
	V/C .... ----
	Slump .... 3.5"
	Age at Test .... 74 days
Cement Portland Cement	Avg. Comp. Strength.. 5210 p.s.i.
	Avg. Flex. Strength.. 1163 p.s.i.
	Avg. Density .... ---- 103.6#/cu.ft.
	Avg. Flex. Strength after test..... ---- 1030 p.s.i.

Beam No.	Symbol	DFE at 300 cycles	No. Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
BS4-1B	———	90.0	———	88.5	———
BS4-1C	— —	90.5	———	88.5	———

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
BS4-1B	Standard	25°-70°F	Sides just slightly attacked.
BS4-1C	Standard	25°-70°F	Same as above.

Project:

FIG. 19

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Concrete Durability Test

Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

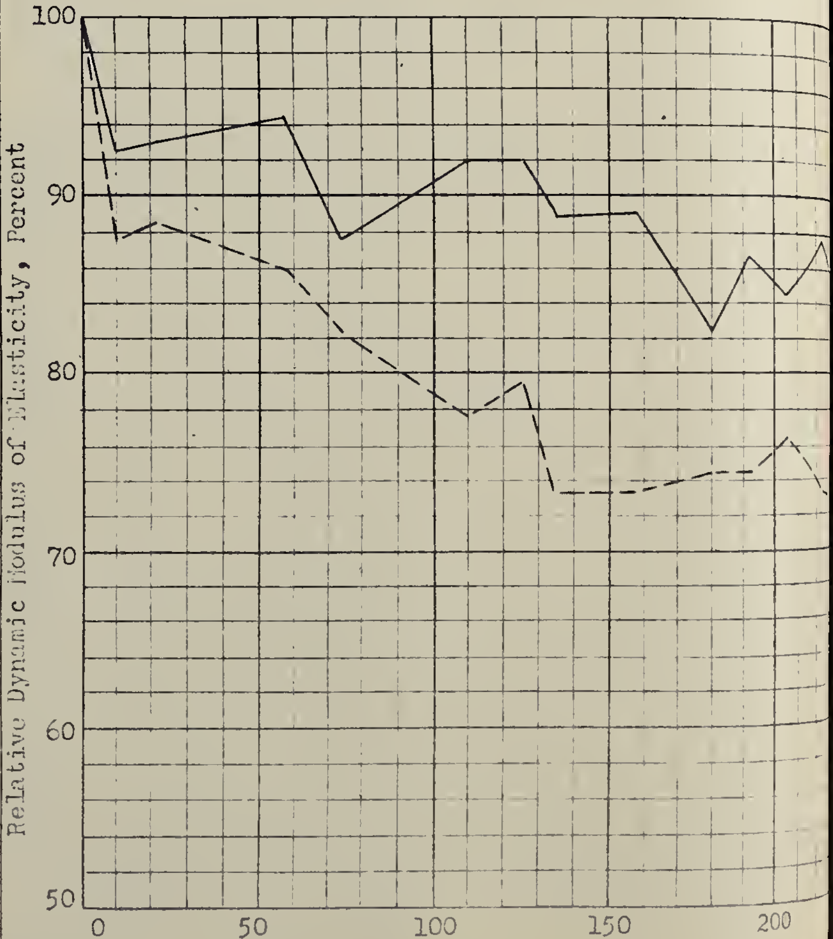
Fine Agg. Smithwick's Fines - 1266# Intermediate - 324#	Mix No. .... 2S Cement Content ...... 669# Water Content .... 334# Air Content .... 5.5% W/C .... 0.50 V/C .... —— Slump .... 3.5" Age at Test .... 82 days Avg. Comp. Strength.. 4215 p.s.i. Avg. Flex. Strength.. 995 p.s.i. Avg. Density .... ——99.4#/cu.ft. Avg. Flex. Strength . after test..... ——995 p.s.i.
Coarse Agg. Smithwick's 482#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.P = 50%
BS5-1A	———	82.5	———	100.0	———
BS5-1C	— —	90.5	———	100.0	———

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
BS5-1A	Standard	25°-70°F	Front badly spalled; edges slightly.
BS5-1C	Standard	25°-70°F	Edges slightly spalled.

Project:

FIG. 20



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerat
freezing and thawing

Project:

FIG. 21

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ility Test

Fine Agg. Smithwick's Fines - 1348# Intermediate - 319#	Mix N Cemer Water Air C W/C . W/C .
Coarse Agg. Smithwick's 473#	Slum Age a Avg. Avg.
Cement Portland Cement	Avg. Avg.

Beam No.	Symbol	DFE at 300 cycles	No. Rel.E
BS6-1A	———	83.6	—
BS6-1D	— —	68.8	—

Beam No.	Method of Curing	Beam Temperature Range
BS6-1A	Standard	25°-70°F
BS6-1D	Standard	25°-70°F

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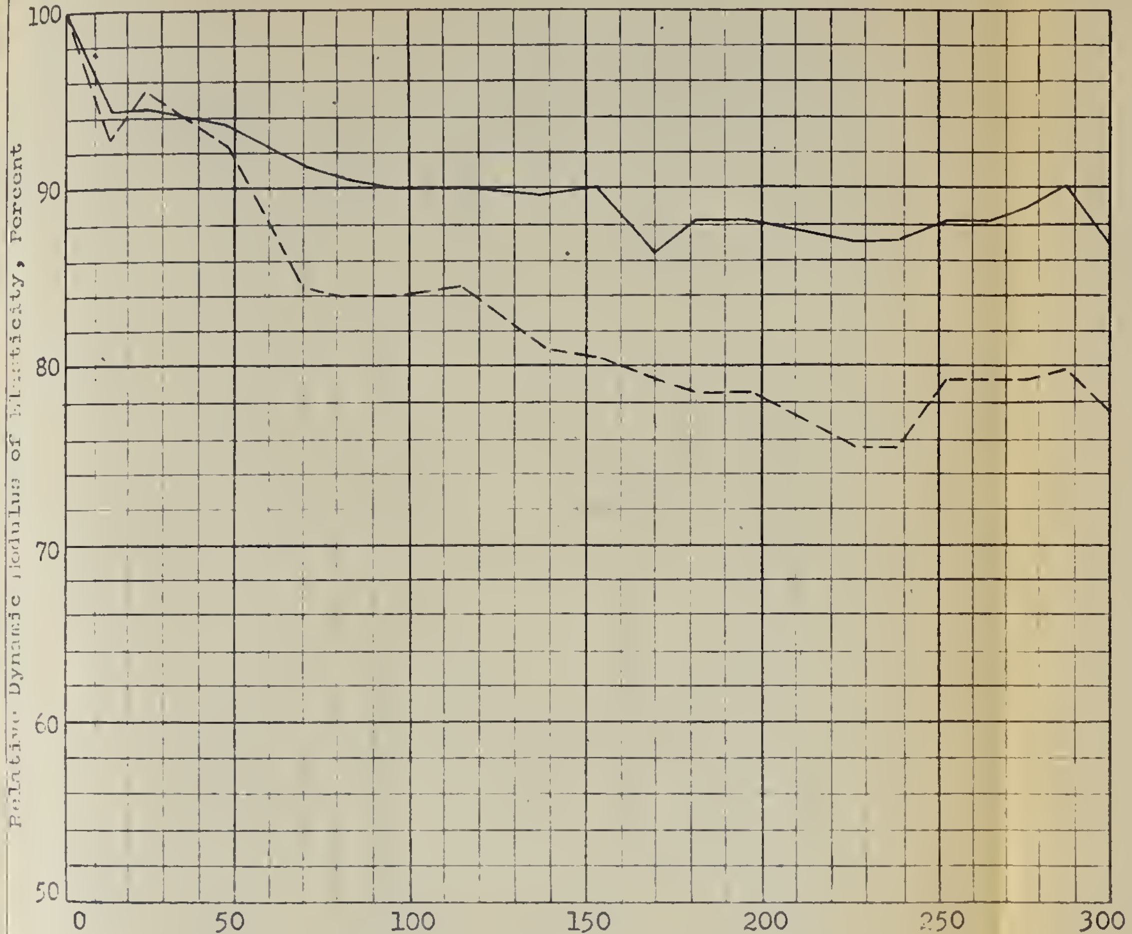
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg. Smithwick's Fines - 1421# Intermediate - 308#	Mix No. .... 4S Cement Content ...... 492# Water Content .... 344# Air Content .... 5.0% W/C .... 0.70 V/C .... —— Slump .... 2.5" Age at Test .... 95 days Avg. Comp. Strength.. 2305 p.s.i. Avg. Flex. Strength.. 808 p.s.i. Avg. Density .... ——98.3#/cu.ft. Avg. Flex. Strength after test..... ——728 p.s.i.
Coarse Agg. Smithwick's 461#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
BS7-1A	———	87.0	———	90.0	———
BS7-1B	— —	77.4	———	90.0	———

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
BS7-1A	Standard	25°-70°F	Edges badly spalled.
BS7-1B	Standard	25°-70°F	Same as above.

Project:

FIG. 22

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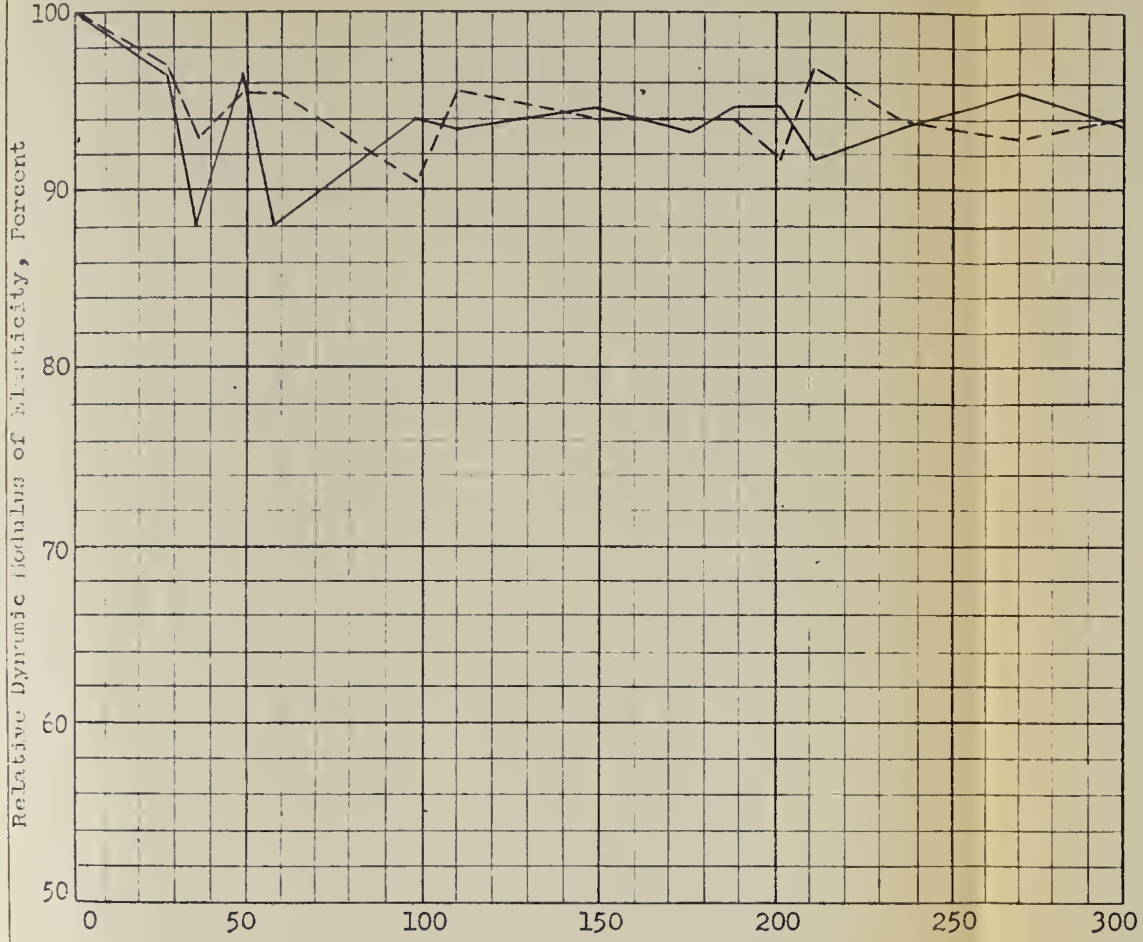
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg. Russell's 1140#	Mix No. .... IR
	Cement Content ...... 822#
	Water Content .... 329#
	Air Content 6.0%
	W/C 0.40
Coarse Agg. Russell's 510#	V/C .... ----
	Slump .... 3"
	Age at Test .... 72 days
	Avg. Comp. Strength.. 3795 p.s.i.
Cement Portland Cement	Avg. Flex. Strength.. 1195 p.s.i.
	Avg. Density .... ---- 93.0#/cu.ft.
	Avg. Flex. Strength after test..... ---- 992 p.s.i.

Beam No.	Symbol	DFE at 300 cycles	No. Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
BR4-IA	———	93.5	------	83.0	------
BR4-IB	— —	94.0	------	83.0	------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
BR4-IA	Standard	25°-70°F	Edges spalled a bit.
BR4-IB	Standard	25°-70°F	Same as above.

Project:

FIG. 23

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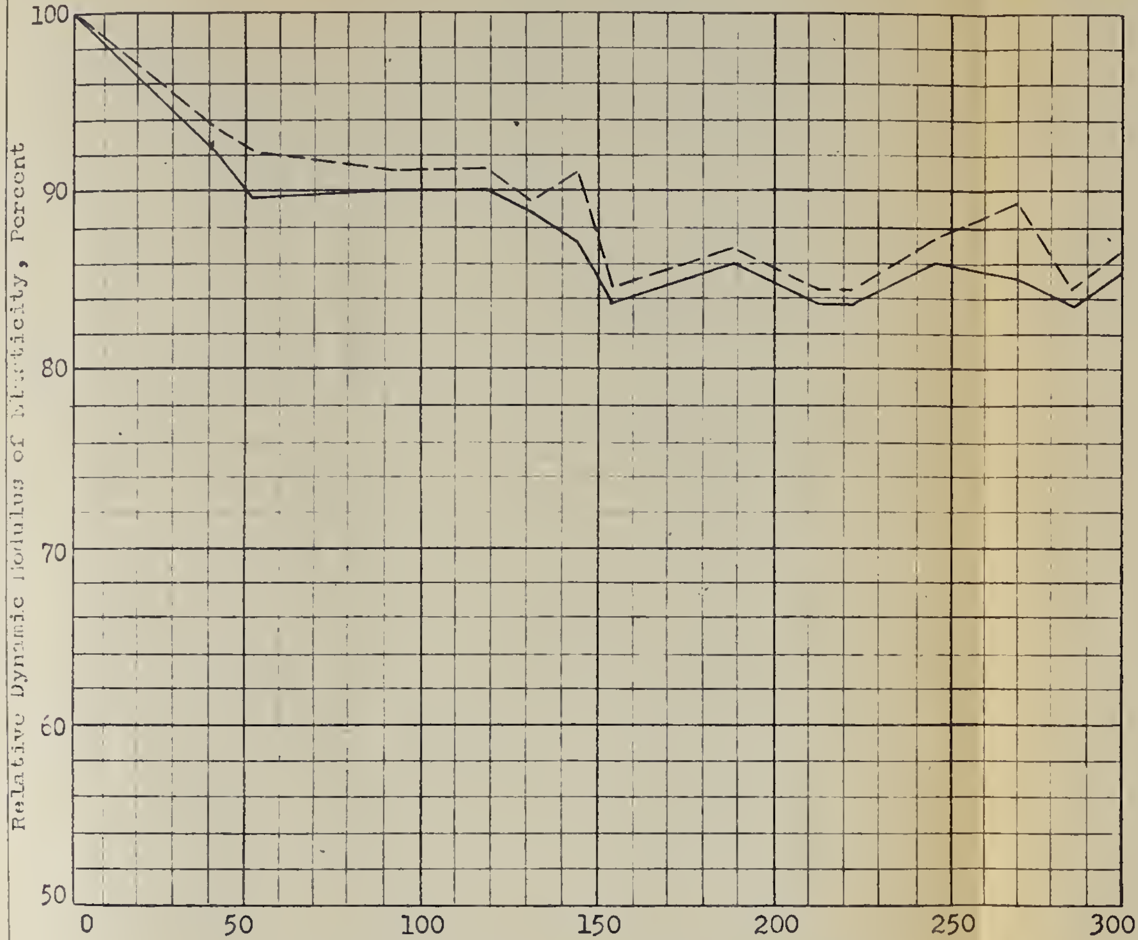
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg. Russell's 1356#	Mix No. 2R
	Cement Content ...... 698#
	Water Content .... 349#
	Air Content 6.0%
Coarse Agg. Russell's 461#	W/C 0.50
	V/C .... ----
	Slump .... 3"
	Age at Test .... 73 days
Cement Portland Cement	Avg. Comp. Strength.. 2975 p.s.i.
	Avg. Flex. Strength.. 905 p.s.i.
	Avg. Density .... ---- 94.2#/cu.ft.
	Avg. Flex. Strength after test..... ---- 895 p.s.i.

Beam No.	Symbol	DFE at 300 cycles	No. Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
BR5-1B	———	85.5	------	99.2	------
BR5-1C	— —	86.6	------	99.2	------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
BR5-1B	Standard	25°-70°F	Sides slightly pitted.
BR5-1C	Standard	25°-70°F	Same as above.

Project:

FIG. 24

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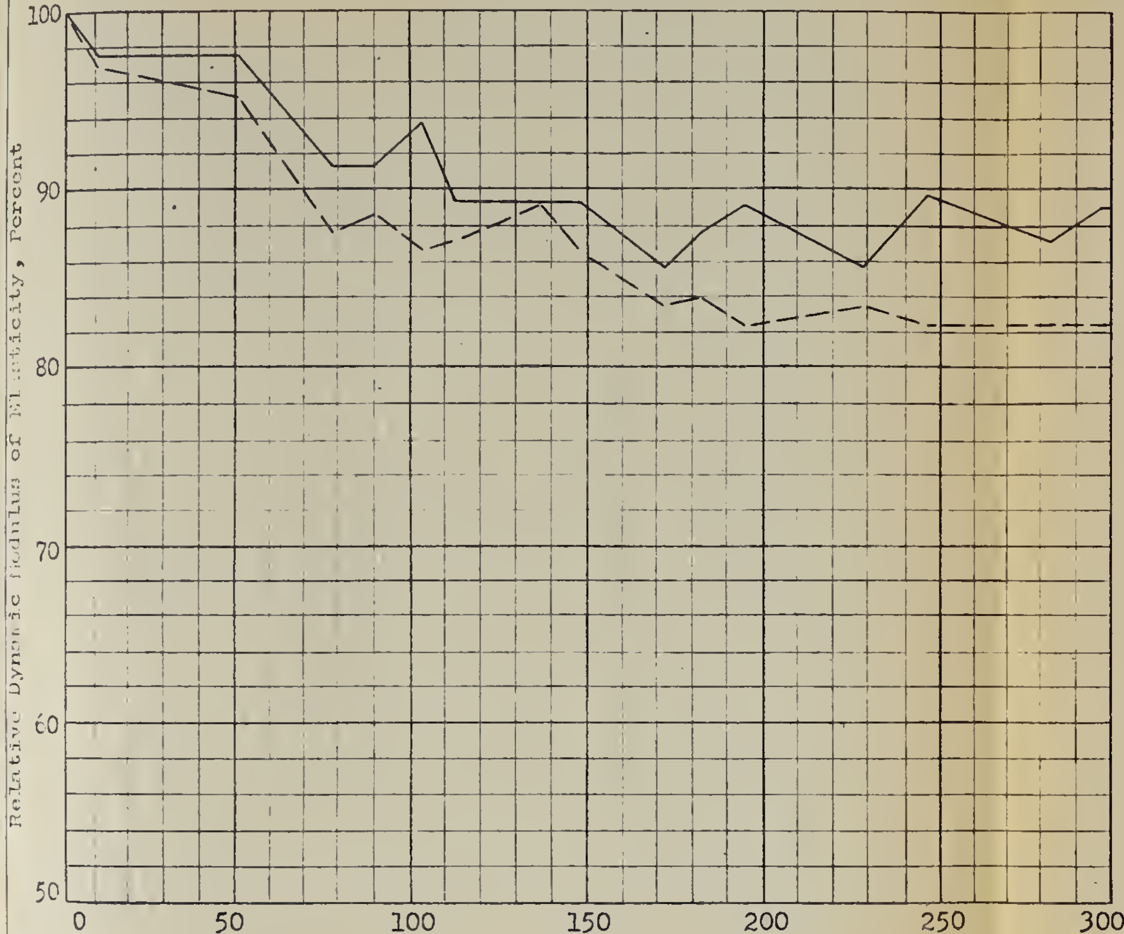
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg. Russell's 1438#	Mix No. .... 3R Cement Content ...... 589# Water Content .... 354# Air Content .... 6.0% W/C .... 0.60 V/C .... ---- Slump .... 4" Age at Test .... 76 days Avg. Comp. Strength.. 1940 p.s.i. Avg. Flex. Strength.. 808 p.s.i. Avg. Density .... ---- 95.7#/cu.ft. Avg. Flex. Strength after test..... ---- 800 p.s.i.
Coarse Agg. Russell's 456#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No. Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
BR6-1A	———	89.0	————	98.9	————
BR6-1B	— —	82.2	————	98.9	————

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
BR6-1A	Standard	25°-70°F	Edges spalled & surface pitted a bit.
BR6-1B	Standard	25°-70°F	Same as above.

Project:

FIG. 25

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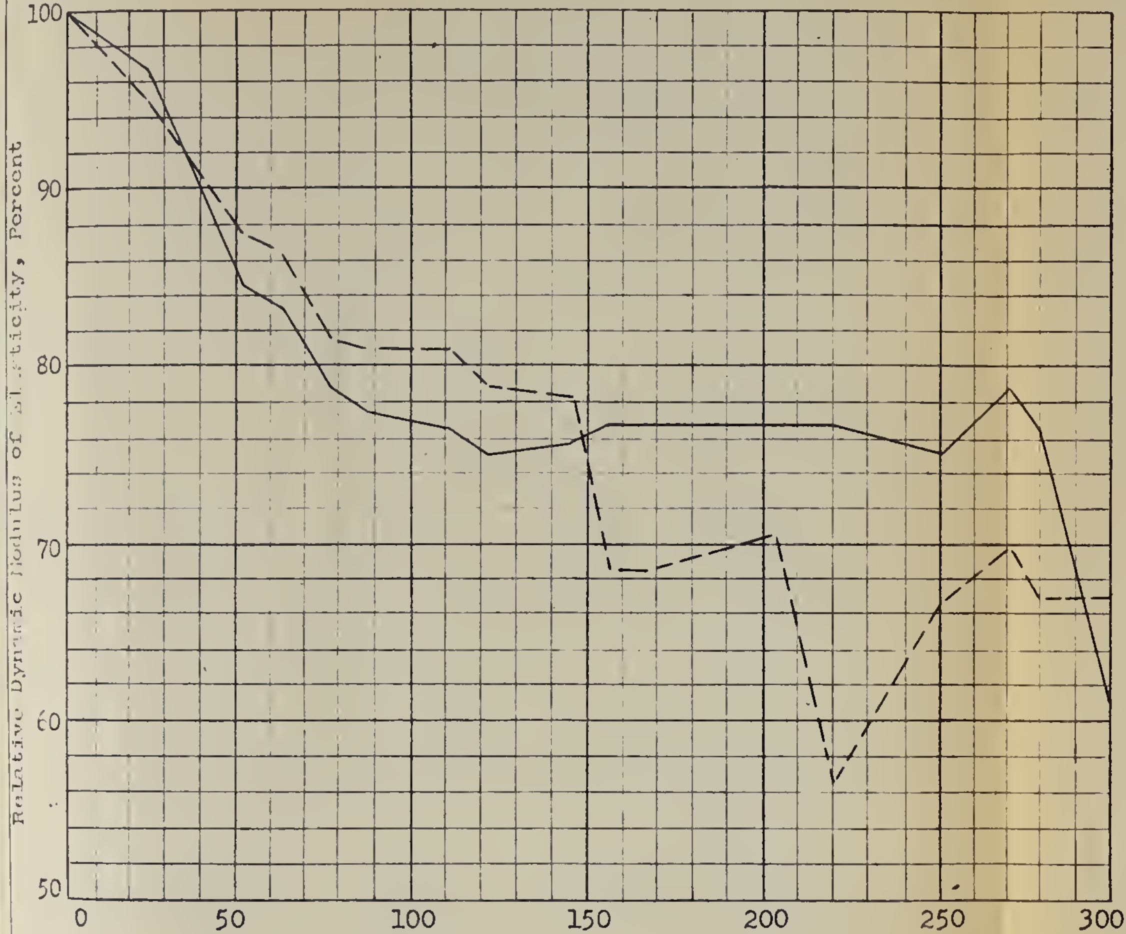
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg. Russell's 1512#	Mix No. .... 4R Cement Content ...... 513# Water Content .... 359# Air Content .... 5.5% W/C .... 0.70 V/C .... ---- Slump .... 3.5" Age at Test .... 73 days Avg. Comp. Strength.. 1140 p.s.i. Avg. Flex. Strength.. 620 p.s.i. Avg. Density .... ---- 93.6#/cu.ft. Avg. Flex. Strength after test..... ---- 418 p.s.i.
Coarse Agg. Russell's 440#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No. Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
BR7-1A	———	61.0	-------	67.4	-------
BR7-1D	— — —	67.0	-------	67.4	-------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
BR7-1A	Standard	25°-70°F	Edges badly spalled. Surface pitted extensively.
BR7-1D	Standard	25°-70°F	Same as above.

Project:

FIG. 26

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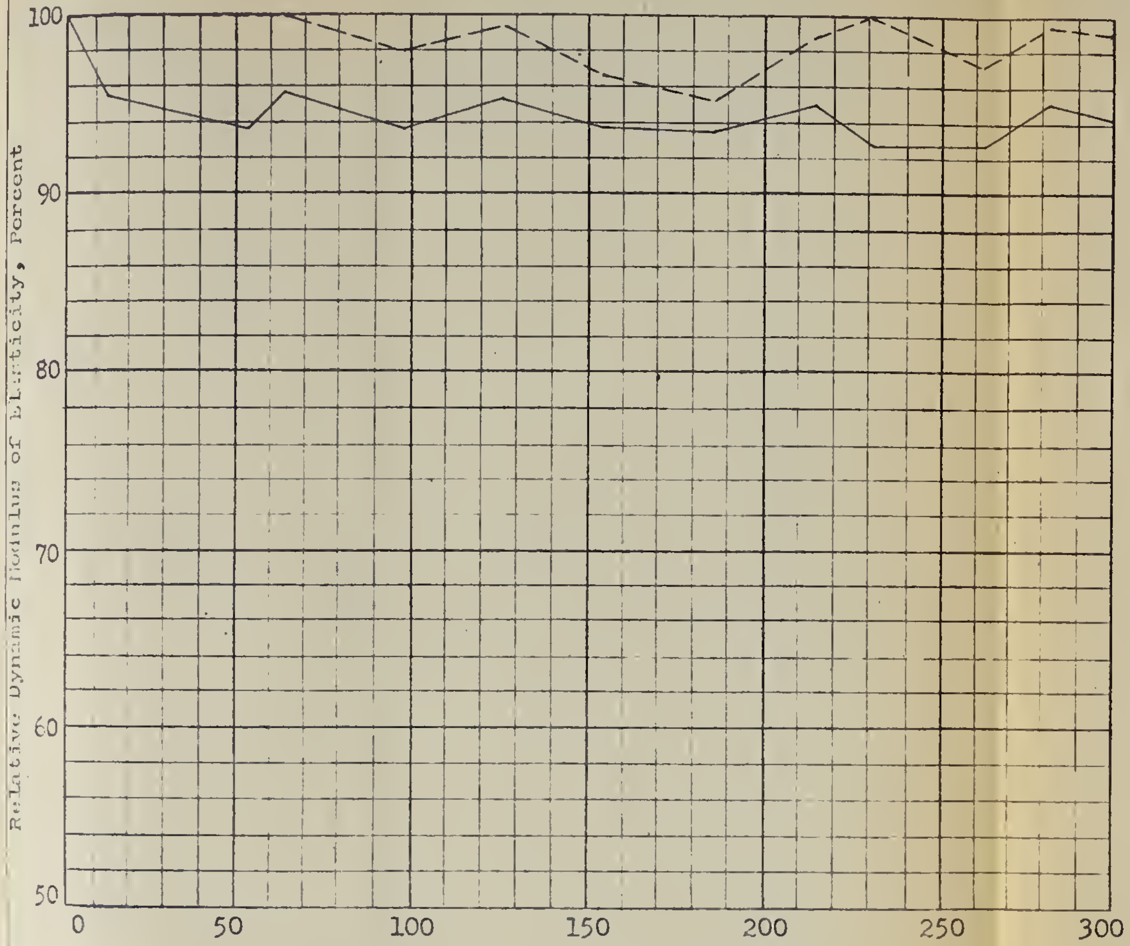
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg. [illegible]	Mix No. .... [illegible] Cement Content ...... [illegible] Water Content .... [illegible] Air Content .... [illegible] W/C .... [illegible] V/C .... [illegible] Slump .... [illegible] Age at Test .... [illegible] Avg. Comp. Strength.. [illegible] Avg. Flex. Strength.. [illegible] Avg. Density .... 148.0#/cu. ft. Avg. Flex. Strength after test..... 1139 p.s.i.
Coarse Agg. [illegible]	
Cement [illegible]	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
[illegible]	———	[illegible]	--------	98.4	--------
[illegible]-1B	— — —	[illegible]	--------	98.4	--------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]-1B	[illegible]	[illegible]	[illegible]

Project: [illegible]

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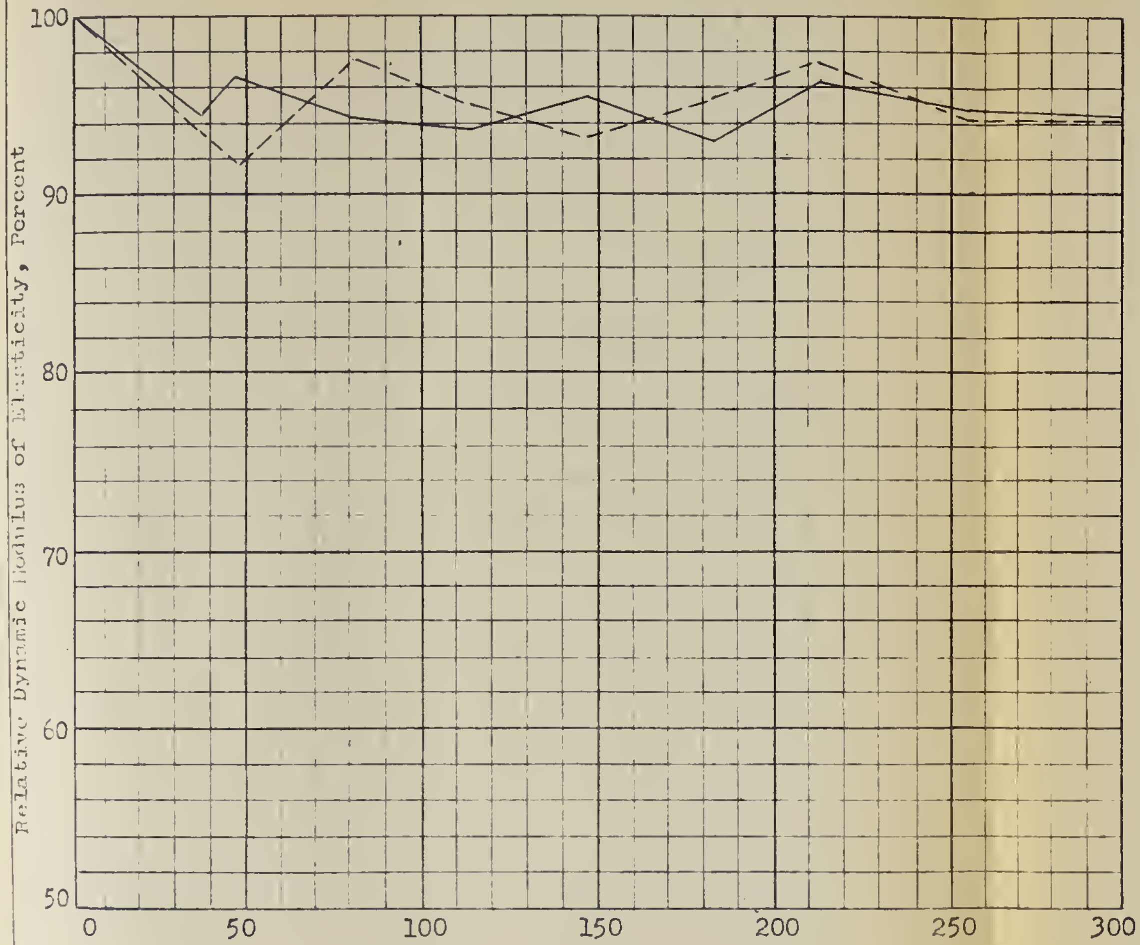
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg. [illegible]	Mix No. .... [illegible] Cement Content ...... [illegible] Water Content .... [illegible] Air Content .... [illegible] W/C .... [illegible] V/C ....---- Slump .... [illegible] Age at Test .... [illegible] Avg. Comp. Strength.. [illegible] Avg. Flex. Strength.. [illegible] p.s.i. Avg. Density ....---- 149.9#/cu.ft. Avg. Flex. Strength . after test.....---- 787 p.s.i.
Coarse Agg. [illegible]	
Cement [illegible]	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.P. = 50%
[illegible]-1		[illegible]	---------	100.0	--------
[illegible]-1D		[illegible]	---------	100.[illegible]	--------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
AB5-1C	[illegible]	25°-70°F	Beam in excellent condition
[illegible]	Standard	25°-70°F	Same as above.

Project: [illegible]

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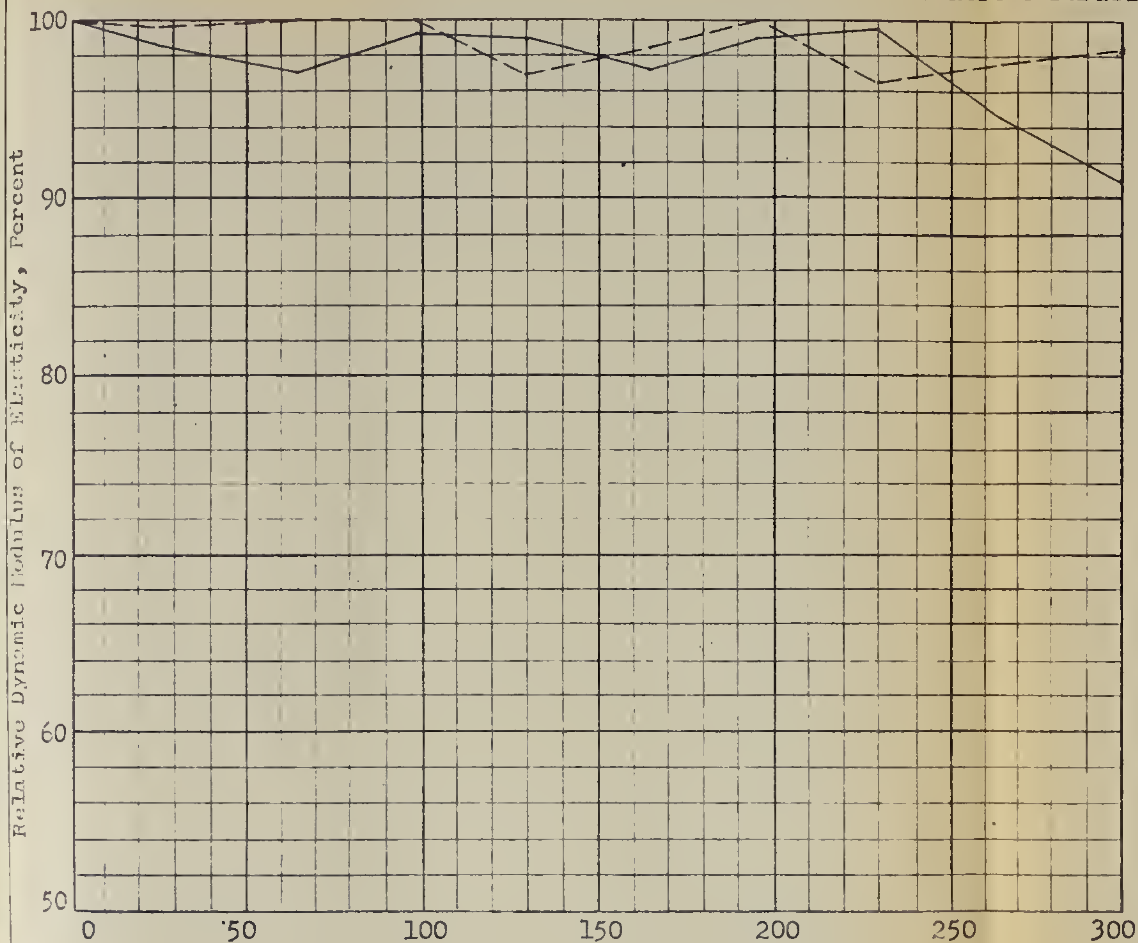
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg. Elk Island Sand 1543#	Mix No. .... A[illegible] Cement Content ...... 4[illegible]# Water Content .... 271# Air Content .... 4.0% W/C .... 0.60 V/C .... ---- Slump .... [illegible] Age at Test .... [illegible]7 days Avg. Comp. Strength.. [illegible]30 p.s.i. Avg. Flex. Strength.. [illegible]10 p.s.i. Avg. Density .... ---- 146.3#/cu.ft. Avg. Flex. Strength after test..... ---- 810 p.s.i.
Coarse Agg. Horse Hills 178[illegible]#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.P = 50%
A[illegible]-1A	———	90.8	-------	100.0	-------
AB[illegible]-1C	— — —	98.[illegible]	-------	100.0	-------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
A[illegible]-1A	Standard	[illegible]°-70°F	Beam in excellent condition.
A[illegible]-1C	Standard	25°-70°F	Same as above.

Project:

FIG. [illegible]9

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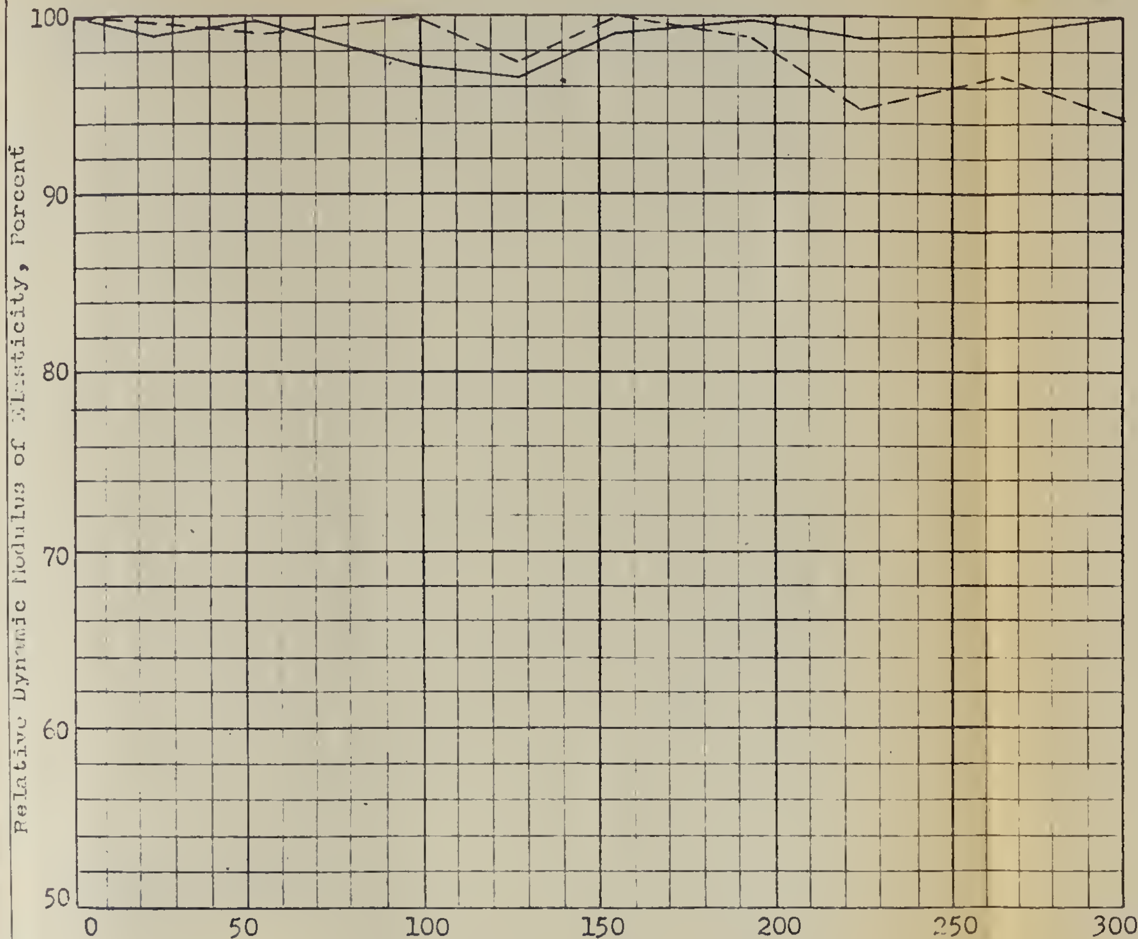
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg.	Mix No. ....
	Cement Content ......
	Water Content ....
	Air Content ....
Coarse Agg.	W/C ....
	V/C .... ----
	Slump ....
	Age at Test ....
Cement	Avg. Comp. Strength..
	Avg. Flex. Strength..
	Avg. Density .... ---- 141.5#/cu.ft.
	Avg. Flex. Strength after test..... ---- 654 p.s.i.

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.P = 50%
[illegible]	———	100.	--------	[illegible]	-------
[illegible]	— — —	[illegible]	--------	100.	-------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]

Project:

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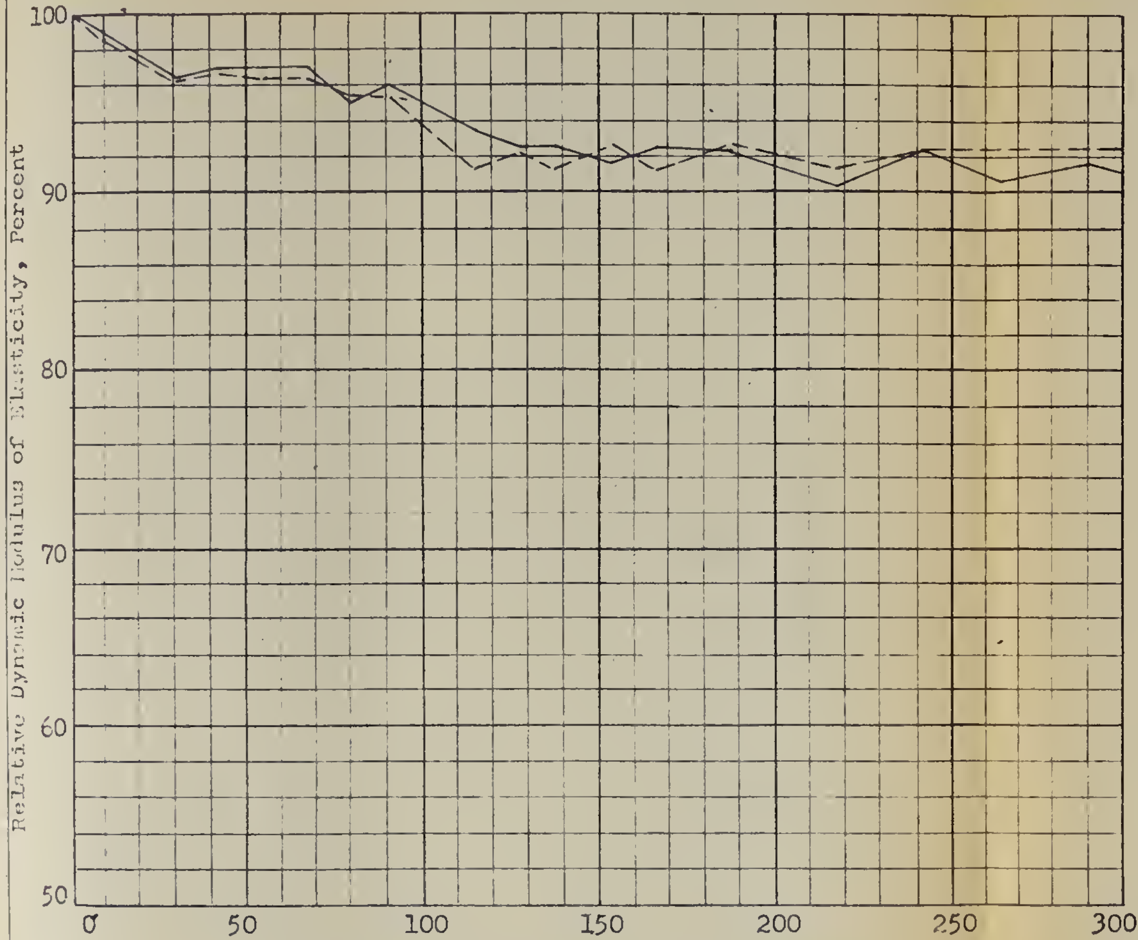
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg.	
Smithwick's 950#	Mix No. ....A.1 Cement Content ......663# Water Content ....265# Air Content ....?.0% W/C ....0.40 V/C ....---- Slump ....?.5" Age at Test ....79 days Avg. Comp. Strength..41?5 p.s.i. Avg. Flex. Strength..1045 p.s.i. Avg. Density ....---- 107.6#/cu.ft. Avg. Flex. Strength . after test.....---- 863 p.s.i.
Coarse Agg. Smithwick's Intermediate - 360# Course - 580#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
ASB4-1A	————	90.7	---------	92.6	----------
ASB4-1B	— — —	9[illegible]	---------	8[illegible].6	----------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
ASB4-1A	Standard	25°-70°F	Beam in good condition.
ASB4-1B	Standard	25°-70°F	Same as above.

Project:

FIG. 31

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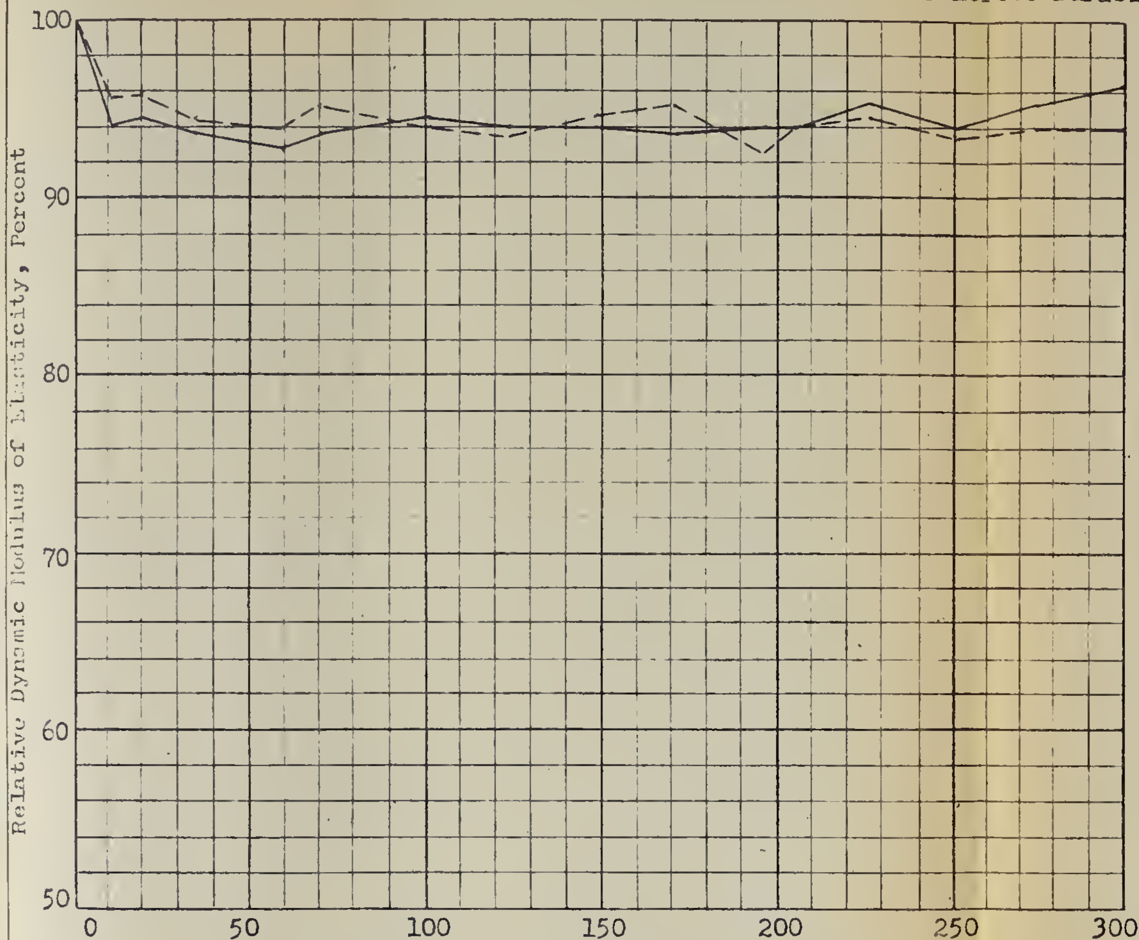
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg. Smithwick's 1025#	Mix No. ....AS5 Cement Content ......541# Water Content ....271# Air Content ....9.5% W/C ....0.50 V/C ....---- Slump ....3" Age at Test ....83 da. Avg. Comp. Strength..3100 p.s.i. Avg. Flex. Strength..770 p.s.i. Avg. Density ....---- 95.0#/cu.ft. Avg. Flex. Strength after test.....---- 769 p.s.i.
Coarse Agg. Smithwick's Intermediate - 350# Coarse - 578#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.P = 50%
AsB5-1A		96.3	-------	99.7	---------
AsB5-1D		94.0	-------	99.7	---------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
AsB5-1A	Standard	25°-70°F	Beam in good condition.
AsB5-1D	Standard	25°-70°F	Same as above.

Project:

FIG. 32

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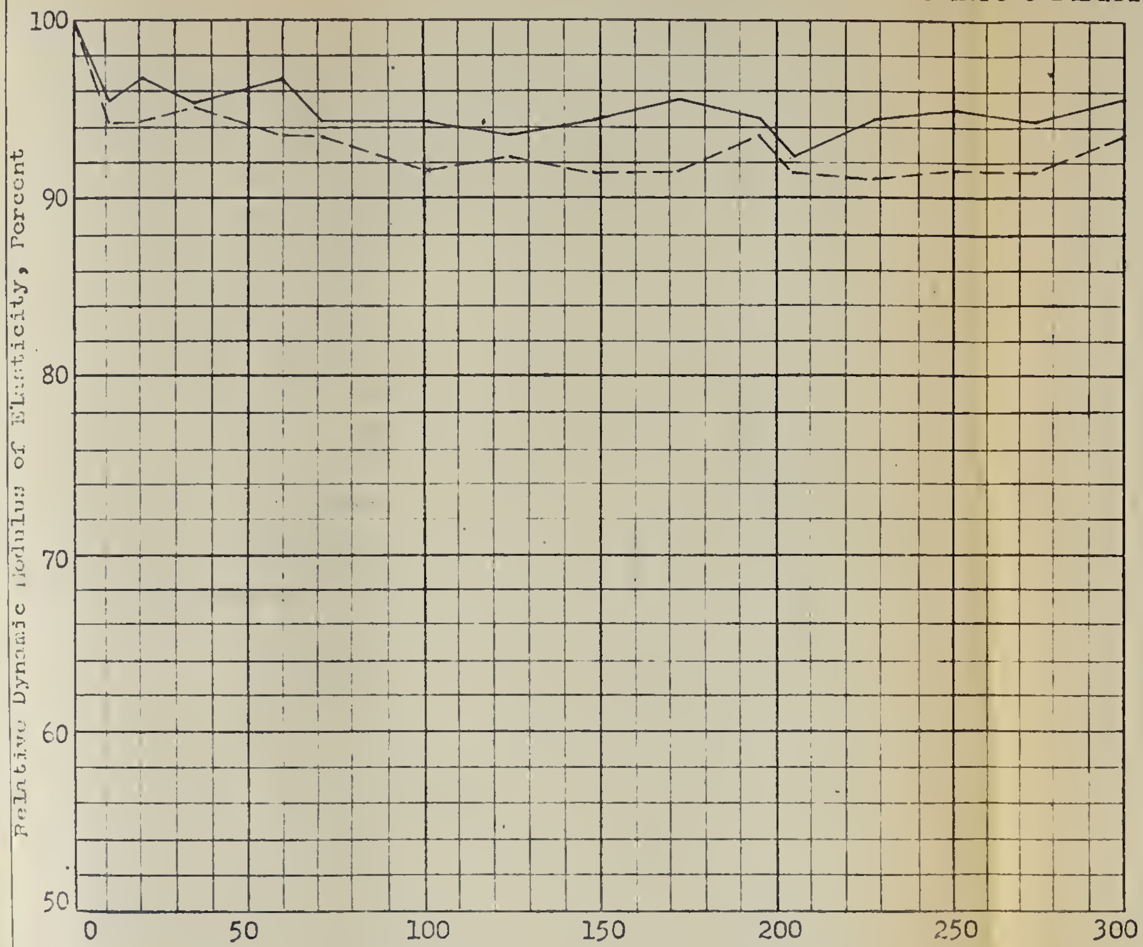
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg.	
Smithwick's 1092#	Mix No. AS3
	Cement Content ......450#
	Water Content ....276#
Coarse Agg. Smithwick's Intermediate - [illegible]# Coarse - 567#	Air Content 10.0%
	W/C ....[illegible]
	V/C ....----
	Slump ....7.5"
Cement Portland Cement	Age at Test ....32 days
	Avg. Comp. Strength..[illegible]90 p.s.i.
	Avg. Flex. Strength..323 p.s.i.
	Avg. Density ....---- 92.6#/cu.ft.
	Avg. Flex. Strength after test.....---- 689 p.s.i.

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.P = 50%
ASB6-1B	————	95.6	--------	74.6	----------
ASB6-1C	— — —	95.[illegible]	--------	74.[illegible]	----------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
ASB6-1B	Standard	65°-70°F	Beam in good condition.
ASB6-1C	Standard	65°-70°F	Same as above.

Project:

FIG. 53

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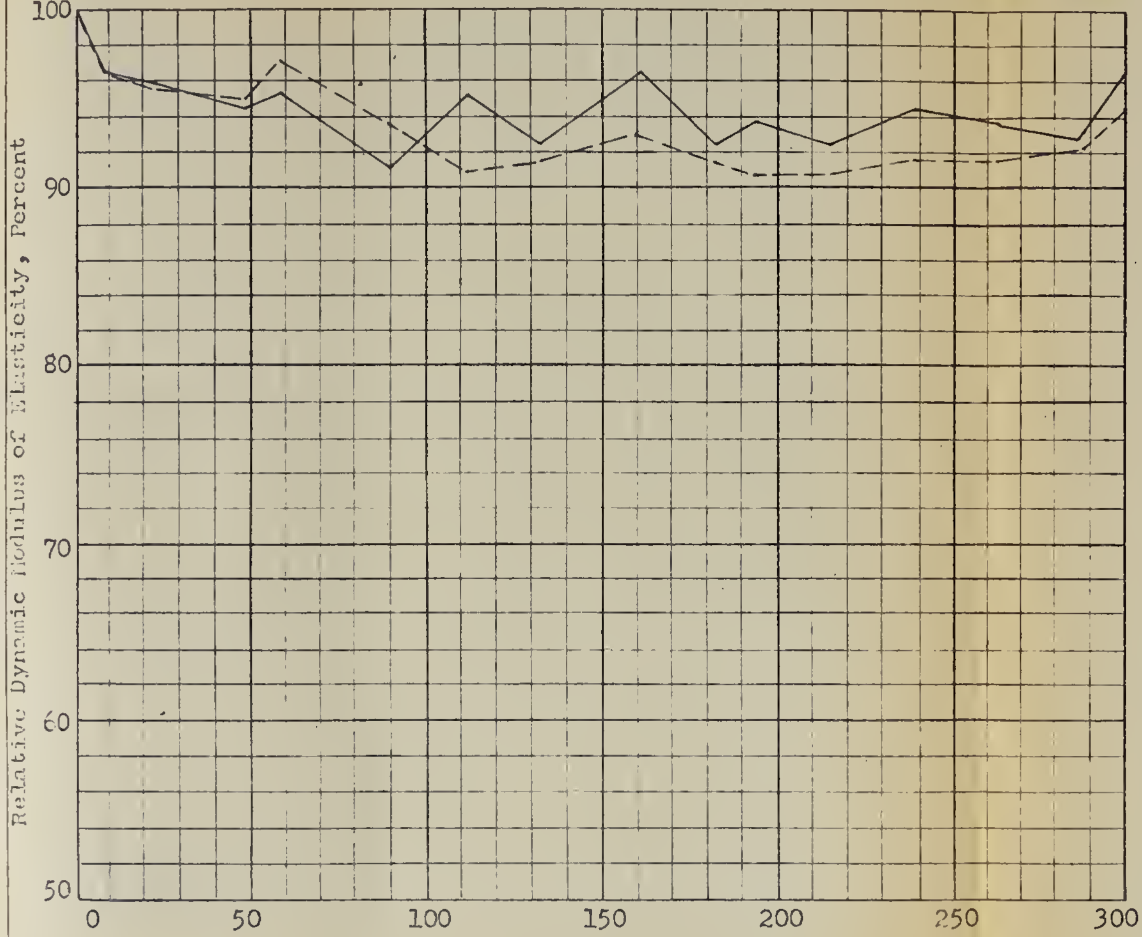
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg. Smithwick's 1135#	Mix No. .... [illegible] Cement Content ......401# Water Content ....281# Air Content ....10.0% W/C ....0.70 V/C ....---- Slump ....3.5" Age at Test ....[illegible] days Avg. Comp. Strength.. 170 p.s.i. Avg. Flex. Strength..[illegible] p.s.i. Avg. Density ....---- 89.4#/cu.ft. Avg. Flex. Strength . after test.....---- 815 p.s.i.
Coarse Agg. Smithwick's Intermediate - [illegible]% Coarse - [illegible]%	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
ASB7-1A	———	96.[illegible]	--------	99.9	--------
ASB7-1C	— — —	94.3	--------	99.9	--------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
ASB7-1A	Standard	[illegible]°-70°F	Beam in good condition.
ASB7-1C	Standard	25°-70°F	Same as above.

Project:

FIG. 34

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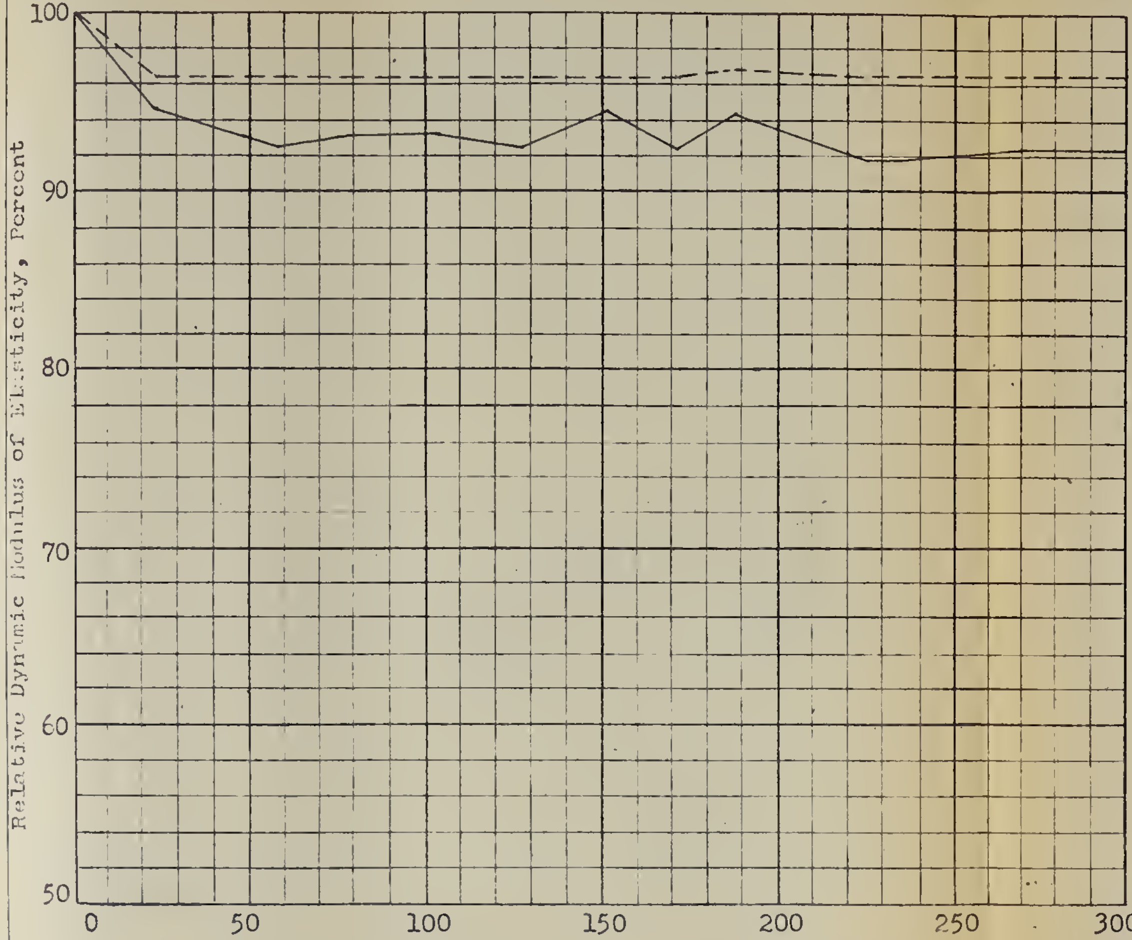
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg.	Mix No. .... [illegible]
[illegible]ell's 1[illegible]#	Cement Content ...... 700#
	Water Content .... [illegible]80#
	Air Content .... 1[illegible].0%
Coarse Agg.	W/C 0.40
Russell's 571#	V/C .... ----
	Slump .... [illegible]
	Age at Test .... 94 days
Cement	Avg. Comp. Strength.. 2890 p.s.i.
Portland Cement	Avg. Flex. Strength.. 730 p.s.i.
	Avg. Density .... ---- 89.1#/cu.ft.
	Avg. Flex. Strength after test..... ---- 520 p.s.i.

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
AR84-1B	———	92.4	---------	71.2	---------
AR34-1D	— — —	96.2	---------	71.2	---------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
AR84-1B	Standard	25°- [illegible]0°F	Beam in good condition.
AR84-1D	Standard	25°-70°F	Same as above.

Project:

FIG. [illegible]5

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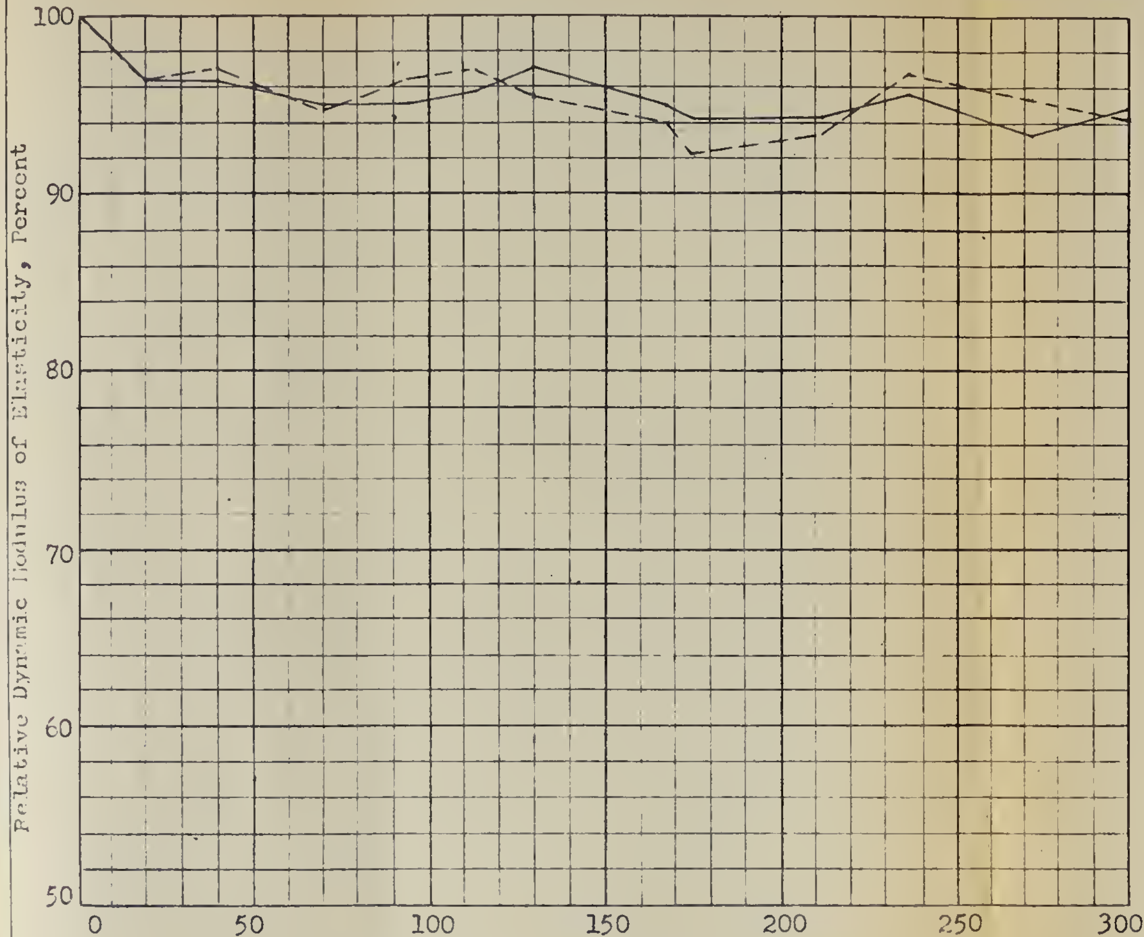
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg. Russell's 100%	Mix No. ....[illegible] Cement Content ......570[illegible] Water Content ....[illegible] Air Content ....1[illegible].0% W/C ....0.[illegible] V/C ....---- Slump ....3" Age at Test ....9[illegible] Avg. Comp. Strength..1915 p.s.i. Avg. Flex. Strength..639 p.s.i. Avg. Density ....---- 83.5#/cu.ft. Avg. Flex. Strength . after test.....---- 639 p.s.i.
Coarse Agg. Russell's [illegible]	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
ARB5-1C	———	95.0	--------	100.0	-------
ARB5-1D	— — —	94.1	--------	100.0	-------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
ARB5-1C	Standard	25°-70°F	Beam in good condition.
ARB5-1D	Standard	25°-70°F	Same as above.

Project: FIG. 36

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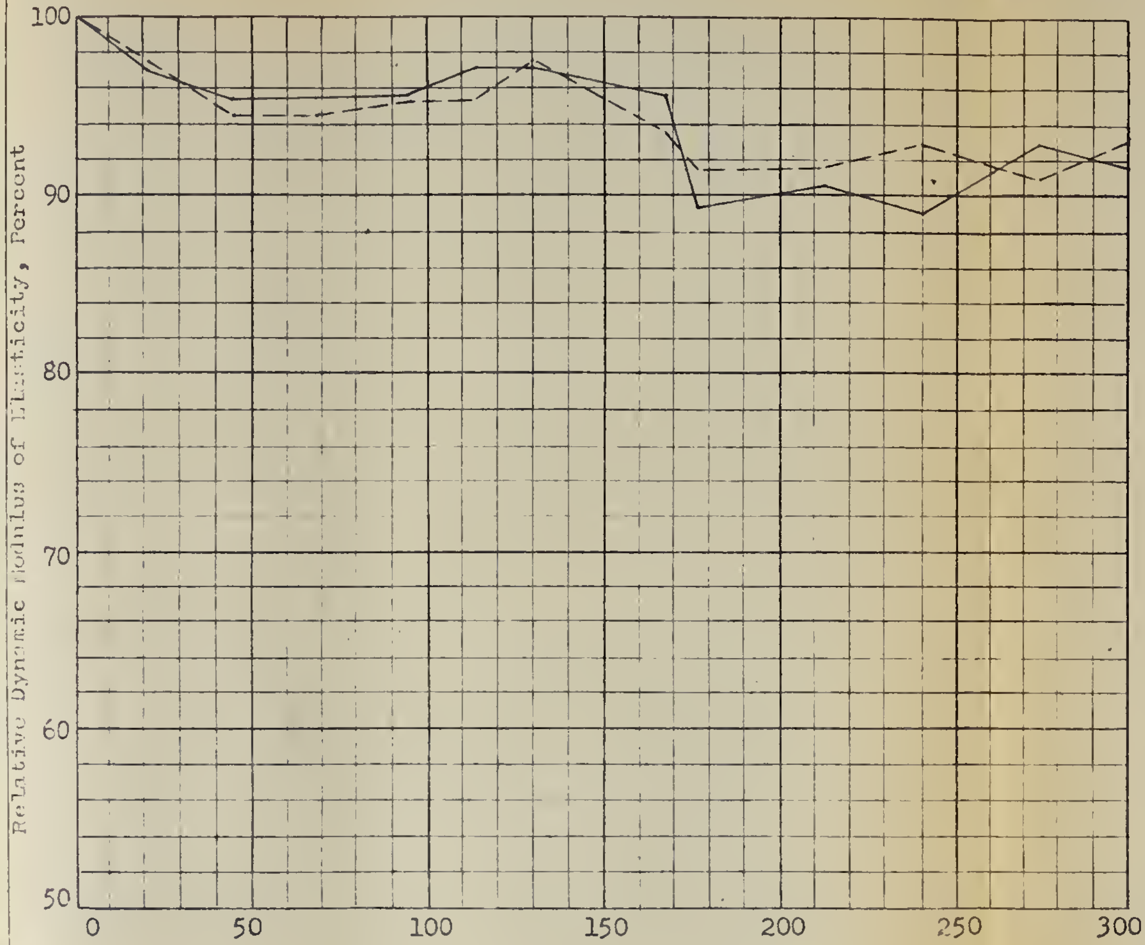
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated freezing and thawing

Fine Agg.	
Russell's 1170#	Mix No. ....[illegible]
	Cement Content ......492#
	Water Content .... 90#
	Air Content 11.0%
Coarse Agg.	W/C 0.60
Russell's 558#	V/C ....----
	Slump ....3.0"
	Age at Test ....94 days
Cement	Avg. Comp. Strength..1475 p.s.i.
Portland Cement	Avg. Flex. Strength..616 p.s.i.
	Avg. Density ....---- 84.9#/cu.ft.
	Avg. Flex. Strength after test.....---- 616 p.s.i.

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
ARB6-18	————	91.6	------	100.0	--------
ARB6-19	— — —	93.0	------	100.0	--------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
ARB6-18	Standard	[illegible]°-70°F	Beam in good condition.
ARB6-19	Standard	[illegible]°-70°F	Same as above.

Project:

FIG. 37

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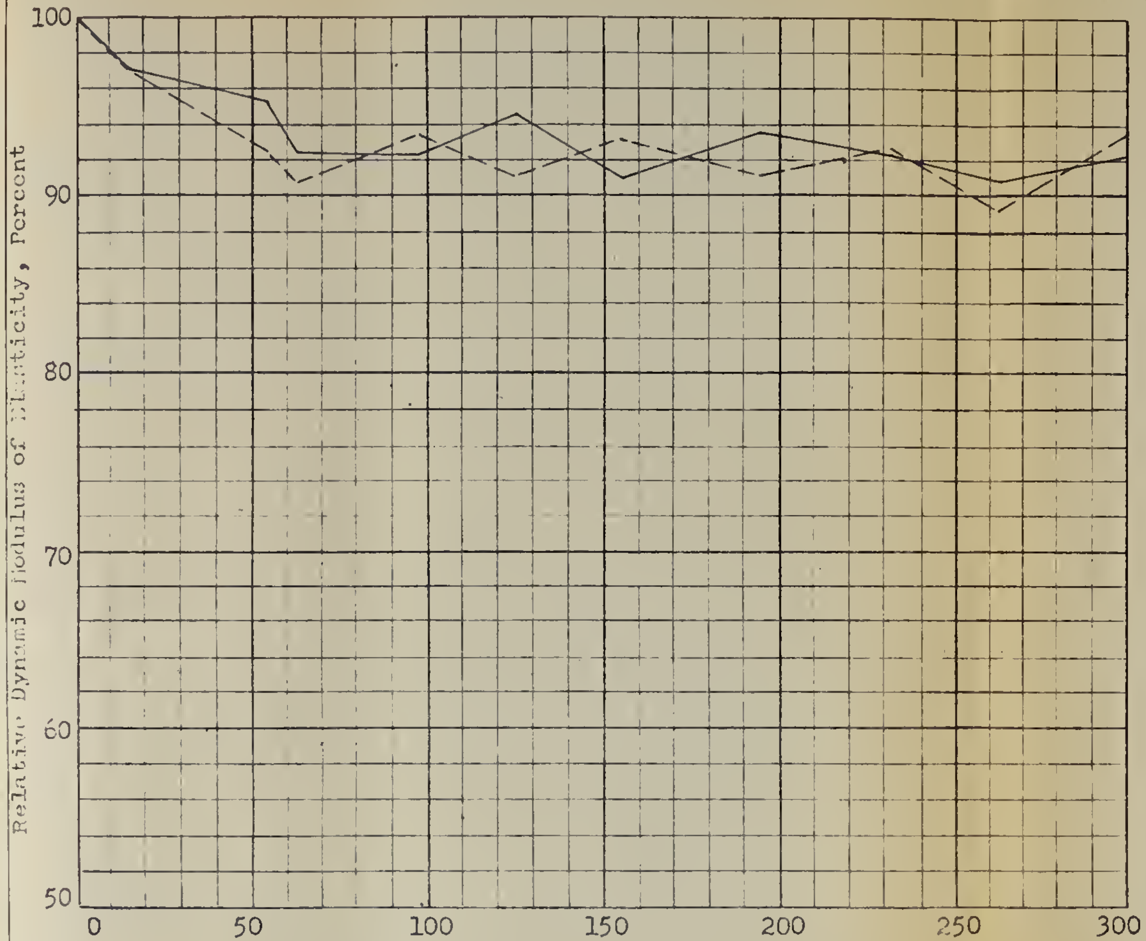
Concrete Durability Test



Two hour cycles of freezing and thawing
Resistance of concrete beams to accelerated
freezing and thawing

Fine Agg. Russell's 1[illegible]2#	Mix No. ....A[illegible] Cement Content ......417# Water Content ....[illegible]95# Air Content ....1[illegible].0[illegible] W/C ....0.[illegible] V/C ....---- Slump ....[illegible]. " Age at Test ....10[illegible] days Avg. Comp. Strength..111[illegible] p.s.i. Avg. Flex. Strength..[illegible]53 p.s.i. Avg. Density ....---- 78.0#/cu.ft. Avg. Flex. Strength . after test.....---- 453 p.s.i.
Coarse Agg. Russell's [illegible]43#	
Cement Portland Cement	

Beam No.	Symbol	DFE at 300 cycles	No.Cycles Rel.E = 50%	DFR at 300 cycles	No. Cycles Rel.R = 50%
ARB7-1[illegible]	———	92.2	------	100.0	------
ARB7-1[illegible]	— — —	93.5	------	100.0	------

Beam No.	Method of Curing	Beam Temperature Range	Visual Inspection
ARB7-1[illegible]	Standard	25°-70°F	Beam in good condition.
ARB7-1D	Standard	25°-70°F	Same as above.

Project:

FIG. 38

Chapter VII

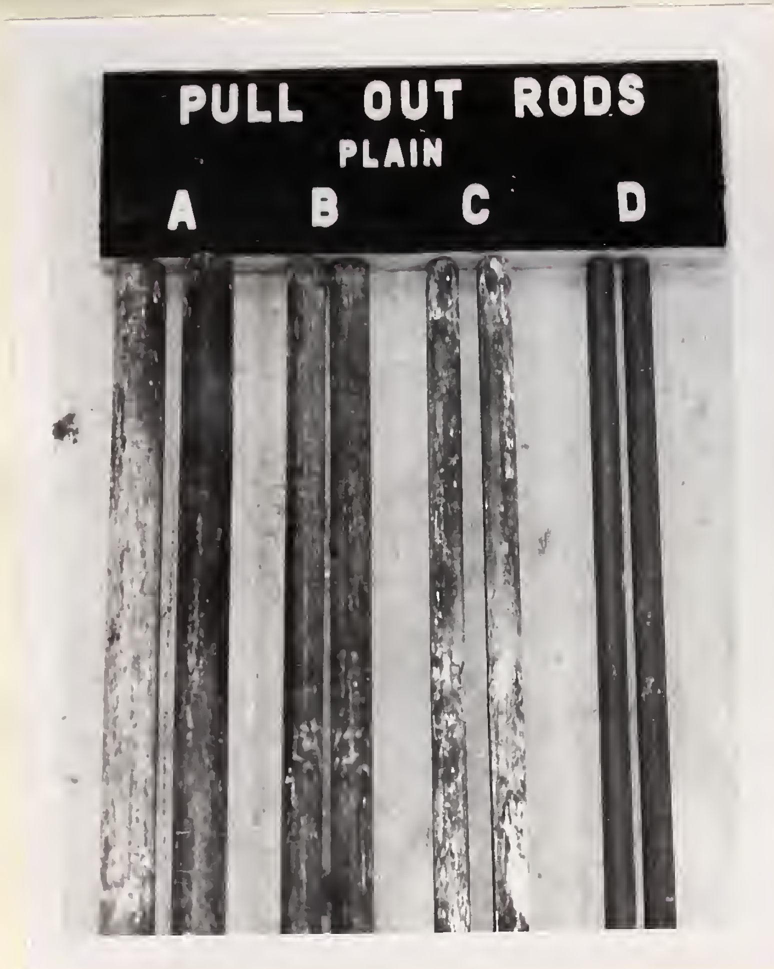
Pull-Out Tests

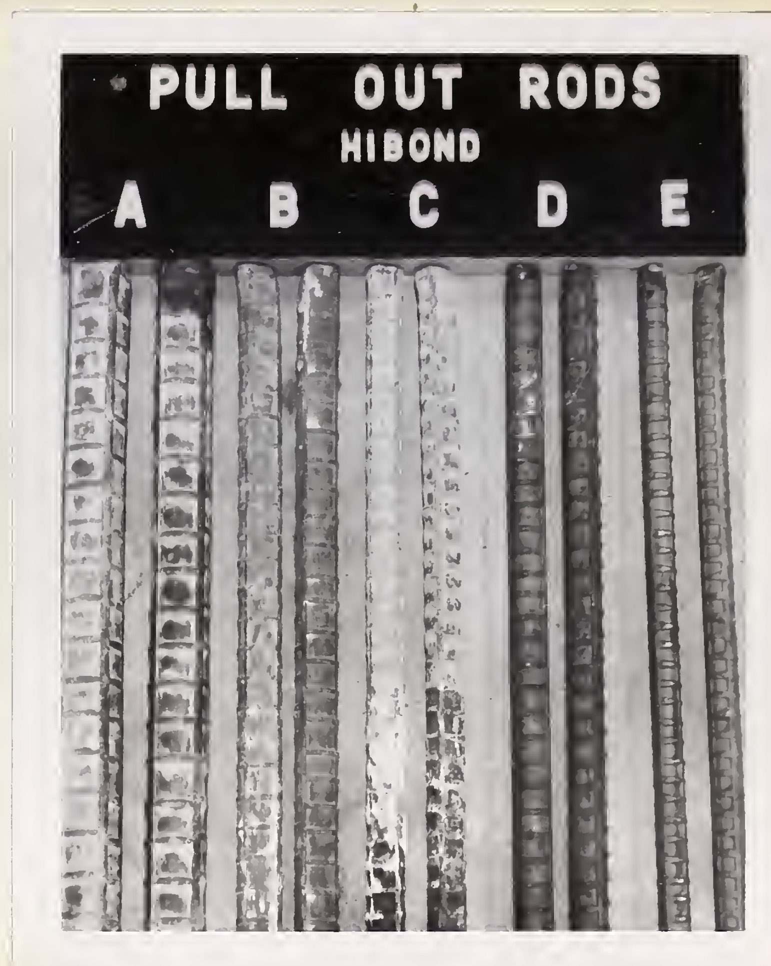
Testing Procedure

As outlined in the general testing program, 288 pull-out tests were conducted. The four w/c ratios used during the previous tests were again used. Plain and hibond rods of 1", 3/4", 5/8" and 1/2" diameters were used (see accompanying photographs). The detail dimensions of the bars are shown in Table 13. In the hibond rods one exception should be noted. The 5/8" diameter hibond series had two rods of different detail dimensions. This was added to see whether or not the ratio of shearing to bearing area had any influence on the results of the bond tests. Three specimens of each bar size, plain and hibond, at each w/c ratio were made up.

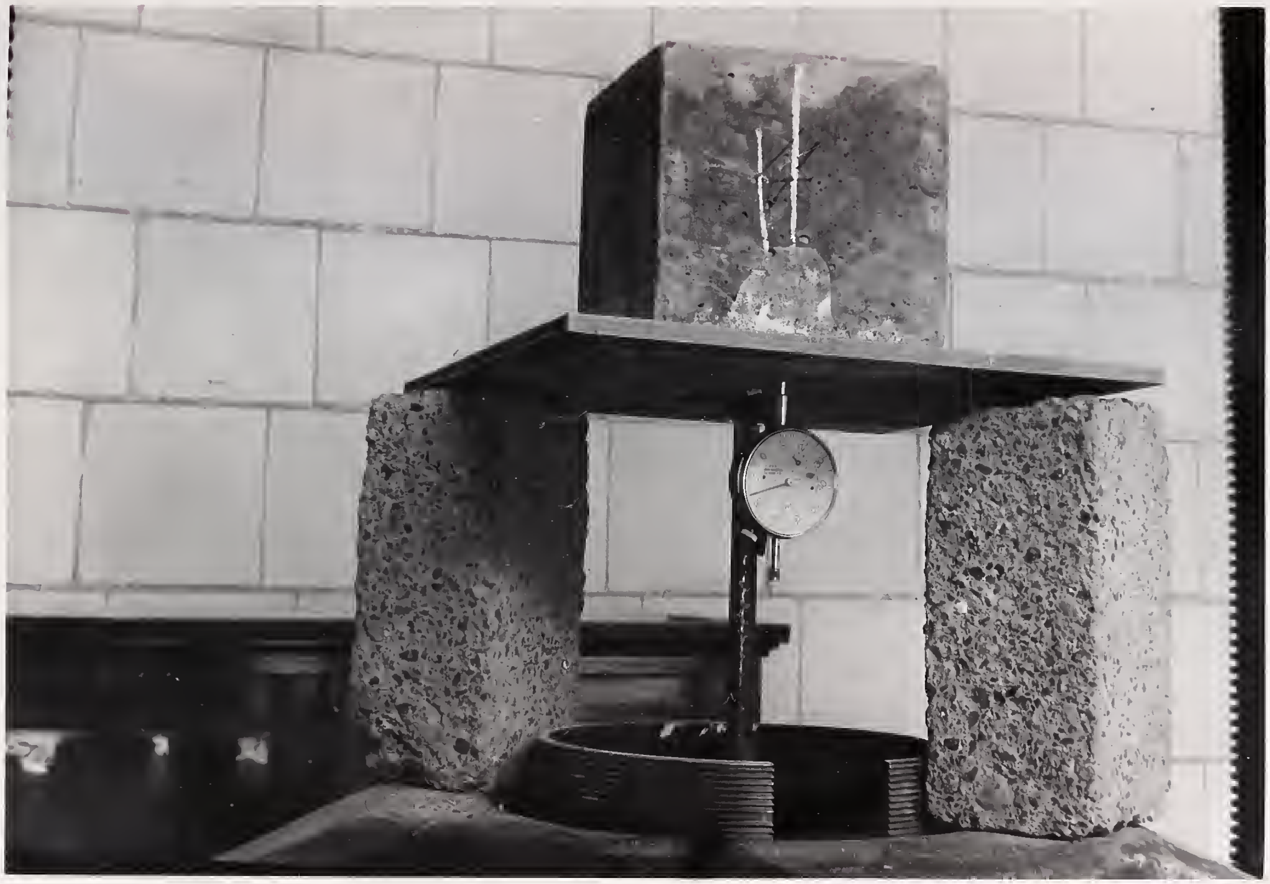
The pull-out specimens were cured for 28 days at room temperature (approximately 70° F.) and moistened sufficiently by canvas hoses to ensure proper curing conditions. Prior to the pull-out tests all the specimens were capped with plaster of paris and allowed to set up. This was to ensure a plain surface on the side upon which the pull was to be exerted. Some difficulty was encountered in making the plaster of paris surface perpendicular to the rod. If this was not the case, it would cause bending of the rod and unsatisfactory readings of the loaded end slippage. Wherever bending did occur the loaded end slip readings were not used. Little difficulty was experienced in obtaining the free end slippage. The only one was due to the sticking of the Ames' dial which happened infrequently.

The pull-out tests were set up as shown in the accompanying photographs. As can be seen the Ames' dial, reading the loaded end slippage, was mounted



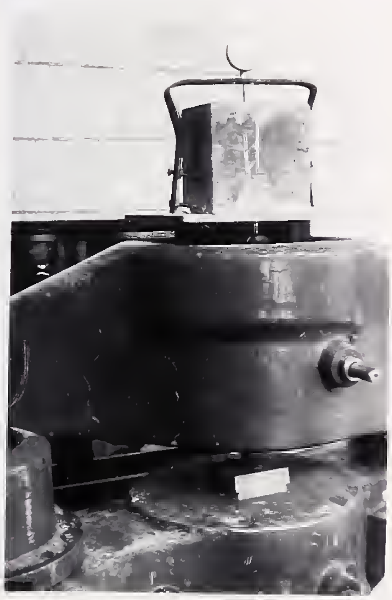


Photographs Nos. 21–22 – Typical Bars used in Pull-Out tests



Photograph No. 23 — Ames' Dial for reading Loaded-End Slip.

as close as possible to the plaster cap. This was done so that no stretching of the steel could take place outside of that portion in the concrete cube. The Ames' dial on the free end was mounted on a clamp which extended over the top of the specimen in place and clamped tightly to the sides. The dial head was in direct contact with the end of the embedded rod.



Photograph No. 24 – Pull–Out Specimen in place for testing.

The pull–out tests were conducted using the Baldwin testing machine. Readings were taken on the Ames' dials and the testing machine to give sufficient results to graph the slip at the loaded and free ends. In the high w/c ratios and plain rod specimens very small increments of load were used. However, because of the small rods it was still difficult to

obtain good readings. The rods failed in bond completely and refused to pick up further load.

Before proceeding further it is appropriate to make a few remarks regarding the validity of a simple pull-out test. The simple pull-out specimen does not involve the combination of forces which come into play in the anchorage zone of a loaded beam it is intended to simulate. This variation is important and has been one of the principal points at issue in arguments of long standing concerning the validity of the pull-out test and the significance of the results obtained from it. The arguments pro and con which have been forwarded are long and varied. The most significant argument probably has been forwarded by Carl A. Menzel. At the 17th semi-annual meeting of the Concrete Reinforcing Steel Institute, in his paper, "A Proposed Standard Deformed Bar for Reinforcing Concrete", (15) Menzel has the following to say:

> "(1) Pull-out tests give a reliable indication of the optimum bond resistance that can be developed with well-designed beam specimens when the concrete in the anchorage zone is effectively reinforced to reduce distortions and cracking in this vital region. Under such conditions the factors which are important in pull-out specimens exert approximately the same relative influence on the bond resistance and anchorage developed in beam specimens.
>
> "(2) Without effective auxiliary reinforcement there is no correlation between the performance of pull-out and beam specimens."

This is not the final word on the subject, however, for Arthur P. Clark in his article, "Bond of Concrete Reinforcing Bars" (16) has this to say:

> "The correlation between the results of the beam and the pull-out tests was such as to indicate that pull-out tests can give reliable estimates of the bonding efficiency of deformed reinforcing bars."

As a result of the inconsistency of reports on these two tests the pull-out test was used to obtain bond results for the concrete under con-

sideration — haydite concrete and sand and gravel concrete.

Test Results

Bond results from pull-out tests are evaluated in four principal manners:

(1) Load-slip curves.

(2) Resistance at slip of 0.010 inches at the loaded end of the bar.

(3) Resistance at slip of 0.001 inches at the free end of the bar.

(4) Maximum resistance or steel stress developed at splitting of the concrete prism in which the bar was embedded or failure completely of the bond.

An attempt was made to present the data in this manner. This became impossible when the variations were noted in the results obtained from the loaded end slip curves.

The pull-out specimens failed in bond except in the low w/c ratios in the hibond bars which failed by splitting of the concrete. All cases where plain bars were used resulted in bond failure.

The results of the pull-out tests are shown in Table 14 and Figures 39 to 56. Figures 39 to 42 are graphs of the load-slip at the loaded end for the 1" diameter hibond bars. They show very little correlation and are typical for all the results at the loaded end. This is no doubt due to not having the plaster of paris cap perpendicular to the embedded rod. On this basis it was decided that the results from the loaded end load-slip curves would be of little value and hence are not presented in Table 14. Figures 39 to 42 are presented, however, to show the variation that was being obtained.

Figures 43 to 50 show the load-slip curves of the free end for the 1" diameter bars. These graphs show very good correlation. It was from

these graphs that the values for resistance at 0.001 inch slip were obtained. The average of the three results is listed in Table 14 for all the various tests. Correlation of the three specimens making up one test was very good as can be seen from the graphs.

Figures 51 to 56 are an attempt to establish a direct relationship, linear if possible, between bond and compressive strength for both the hibond and plain bars. This attempt was not too successful. Good linear relationships exist in some cases, notably Figure 51. Otherwise there seems to be little correlation and little similarity between the curves for the various size bars.

In general, the curves are nearly horizontal at an end slip of 0.01 inches indicating that the bond resistance of the hibond bars used was similar in nature to that of plain bars. At the maximum load either the concrete cube split or the bar merely continued to slip with a decrease in load. The average values of bond stress developed by the pull-out specimens at an end slip of 0.001 inches and at maximum load are shown in Table 14.

The maximum bond strength developed varied from 31.0% to 33.0% of the compressive strength for sand and gravel in the hibond series and from 8.0% to 11.0% in the plain bar series. Smithwick's aggregate concrete had maximum bond strengths varying from 28.0% to 40.0% of the compressive strength for the hibond series and 18.0% to 22.0% for the plain series. Russell's aggregate concrete had maximum bond strengths varying from 42.0% to 73.0% for the hibond series and 35.0% to 45.0% of the compressive strength for the plain rods. These are based on the test results of the one inch diameter bars. A close investigation of the test results for the other size bars reveals the same range of values.

All pull-out specimens were made of concrete in which an air-entraining agent had been used. Its effect on the bond values could not be evaluated in these tests as no parallel non-entrained mixes were run. However, research has been done on this particular topic by Hognestad and Siess[17] in their paper entitled, "Effect of Entrained Air on Bond Between Concrete and Reinforcing Steel". The results of this investigation showed that air contents up to 5% reduced the bond less than the reduction revealed in flexual and compressive strengths of the concrete. However, where more than 5% of air was entrained, bond of horizontal bars was reduced rapidly. This then should be borne in mind when evaluating the results presented in this chapter. The only series affected to the extent where too much air was being entrained was the concrete made using Russell's aggregate. Variations in the air contents were obtained and some were quite high. There does not seem to be any rapid falling off of the bond for these tests. Therefore it is possible that light weight concrete is not affected in the above mentioned manner.

The use of two types of 5/8" diameter hibond bars in Russell's concrete reveals very little about the influence of the ratio of shearing to bearing areas. Sufficient tests to give any indication were not carried out. A comparison of the maximum bond strength and the ultimate compressive strength of the concrete for the two types of bars reveals no conclusive results. To obtain any information in this respect a more detailed investigation would have to be undertaken. There was not too much difference in the two ratios, being 15 and 10 respectively, so that it is not surprising that no differences in bond strength were noted.

Figures 43 to 50 show that failure occurred very soon after an end slip

of 0.005 inches was reached, and in general slipping developed quite rapidly after an end slip of 0.001 inches had been reached.

Discussion of Test Results

The bond strength results indicate that essentially the same design rules for sand and gravel concrete can be applied to light weight concrete. In the light weight results for the high w/c ratios we find that the maximum bond strength is considerably higher based on the ultimate compressive strength of the concrete than for the corresponding w/c ratio mix in the sand and gravel concrete. A comparison, however, between the ultimate compressive strengths shows that there are large differences in these values. This could to some extent explain the high maximum bond strength to ultimate compressive strength percentage. In the lower strength range the percentage should tend to be higher as is indicated by the results. Despite this, however, the middle strength range (2000 - 3000 p.s.i.) of the light weight concrete exhibits higher bond strengths at both 0.001 inches slip and at ultimate bond strength.

Compare the results of concrete possessing a compressive strength of approximately 2300 p.s.i. in the one inch diameter hibond series.

	Comp. Str. p.s.i.	Bond at 0.001" slip p.s.i.	Ultimate Bond p.s.i.
Sand & Gravel	2430	410	802
Smithwick's	2290	610	945
Russell's	2350	500	990

This pattern is followed whenever a comparison on compressive strength is made.

As is noted in "Review of Research in Ultimate Strength of Reinforced Concrete Members" by C. P. Seis[22]

> "It is, however, difficult to interpret the results of tests of this type in terms of ultimate strengths in bond, since the relationship between slip of the bars and failure in bond has not yet been stated quantitatively."[22]

As a result interpretations of bond tests must be made by comparing the bonding characteristics or the bond-slip relations for various conditions. Numerous variables can be evaluated with careful study. A good work dealing with this is Menzel's paper "Some Factors Influencing the Results of Pull-Out Bond Tests".[23] Mention is made of type of bar, surface condition, strength of concrete, length of embedment, bar diameter, depth of concrete beneath the bar, position of casting and stress conditions in the surrounding concrete. Time did not permit an evaluation of any of the above mentioned factors. Because of the position of casting and the fact that a pull-out test was used the number of variables that could have been evaluated are limited.

These results when compared to those obtained by Richart and Jensen[1] are found to be considerably higher. This would be due to the use of the old deformed bars as opposed to the hibond used in the tests throughout this investigation.

The attempt to establish a direct relationship between bond and compressive strength (see Figures 51-56) showed no correlation. As a result it would seem difficult to base a working stress in bond on a percentage of the compressive strength.

TABLE 13

Detail Dimensions of Hibond Bars

Bar Size diameter inches	Bar No.	Bar Area sq.in.	Deformations: Average spacing inches	Deformations: Average Height inches	Deformations: Bearing Area sq.in./in.	Deformations: Shearing Area sq.in./in.	Deformations: Ratio of Shearing to Bearing Area
1	A	0.832	0.675	0.036	0.170	3.315	19.4
3/4	B	0.456	0.519	0.030	0.133	2.395	18.0
5/8	C	0.325	0.428	0.029	0.131	2.020	15.5
*5/8	D	0.368	0.429	0.042	0.198	2.153	10.9
1/2	E	0.232	0.343	0.045	0.206	1.709	8.3
1	A	0.817	Plain – No deformations				
3/4	B	0.440	" " "				
5/8	C	0.315	" " "				
1/2	D	0.192	" " "				

TABLE 14

Bond Pull-Out Test Results

1" Rounds

W/C	Type of Aggregate	Hibond: Bond at End Slip of 0.001 inches p.s.i.	Hibond: Bond at Ultimate Load p.s.i.	Plain: Bond at End Slip of 0.001 inches p.s.i.	Plain: Bond at Ultimate Load p.s.i.	Average Compressive Strength at Time of Test p.s.i.
0.4	S & G	782	1267	360	426	4030
	S	695	1127	388	568	2960
	R	500	990	322	427	2350
0.5	S & G	727	1127	366	373	3700
	S	610	945	314	471	2290
	R	433	582	298	357	801
0.6	S & G	566	927	215	245	3000
	S	292	468	212	303	1660
	R	305	371	211	221	636
0.7	S & G	410	802	187	203	2430
	S	292	455	195	252	1130
	R	245	281	171	186	448

TABLE 14 (Cont'd.)

3/4" Rounds

W/C	Type of Aggregate	Hibond: Bond at End Slip of 0.001 inches p.s.i.	Hibond: Bond at Ultimate Load p.s.i.	Plain: Bond at End Slip of 0.001 inches p.s.i.	Plain: Bond at Ultimate Load p.s.i.	Average Compressive Strength at Time of Test p.s.i.
0.4	S & G	897	1466	393	469	4133
	S	607	1078	408	521	2647
	R	649	928	345	405	1198
0.5	S & G	398	909	335	349	2755
	S	650	816	292	368	1607
	R	563	779	283	332	1113
0.6	S & G	570	1099	–	311	2797
	S	386	578	210	259	1101
	R	212	244	125	130	1303
0.7	S & G	451	772	220	239	2413
	S	264	351	138	156	857
	R	171	200	95	103	613

TABLE 14 (Cont'd.)

5/8" Rounds

W/C	Type of Aggregate	Hibond: Bond at End Slip of 0.001 inches p.s.i.	Hibond: Bond at Ultimate Load p.s.i.	Plain: Bond at End Slip of 0.001 inches p.s.i.	Plain: Bond at Ultimate Load p.s.i.	Average Compressive Strength at Time of Test p.s.i.
0.4	S & G	1085	1587	524	587	3990
	S	1806	1059	475	580	3030
	R	746	999	395	520	1355
0.5	S & G	833	1182	-	377	3220
	S	452	601	282	341	2182
	R	*723	965	443	587	2558
0.6	S & G	818	1272	233	353	2775
	S	335	397	217	223	1485
	R	*522	676	381	470	1325
0.7	S & G	564	979	-	294	2295
	S	223	275	160	192	1088
	R	*345	398	250	282	875

TABLE 14 (Cont'd.)

1/2" Rounds

W/C	Type of Aggregate	Hibond: Bond at End Slip of 0.001 inches p.s.i.	Hibond: Bond at Ultimate Load p.s.i.	Plain: Bond at End Slip of 0.001 inches p.s.i.	Plain: Bond at Ultimate Load p.s.i.	Average Compressive Strength at Time of Test p.s.i.
0.4	S & G	1107	1427	502	618	4362
	S	622	947	287	470	3027
	R	715	892	340	380	1662
0.5	S & G	903	1240	400	456	3310
	S	470	577	282	316	2120
	R	746	827	478	543	1740
0.6	S & G	665	1015	–	309	2605
	S	284	356	115	152	1192
	R	648	827	459	539	1555
0.7	S & G	672	930	242	251	2450
	S	–	234	85	101	722
	R	533	614	264	303	1077

FIG. 39

LOADED END SLIP CURVES- 1" DIA. HIBOND BARS W/C = 0.40

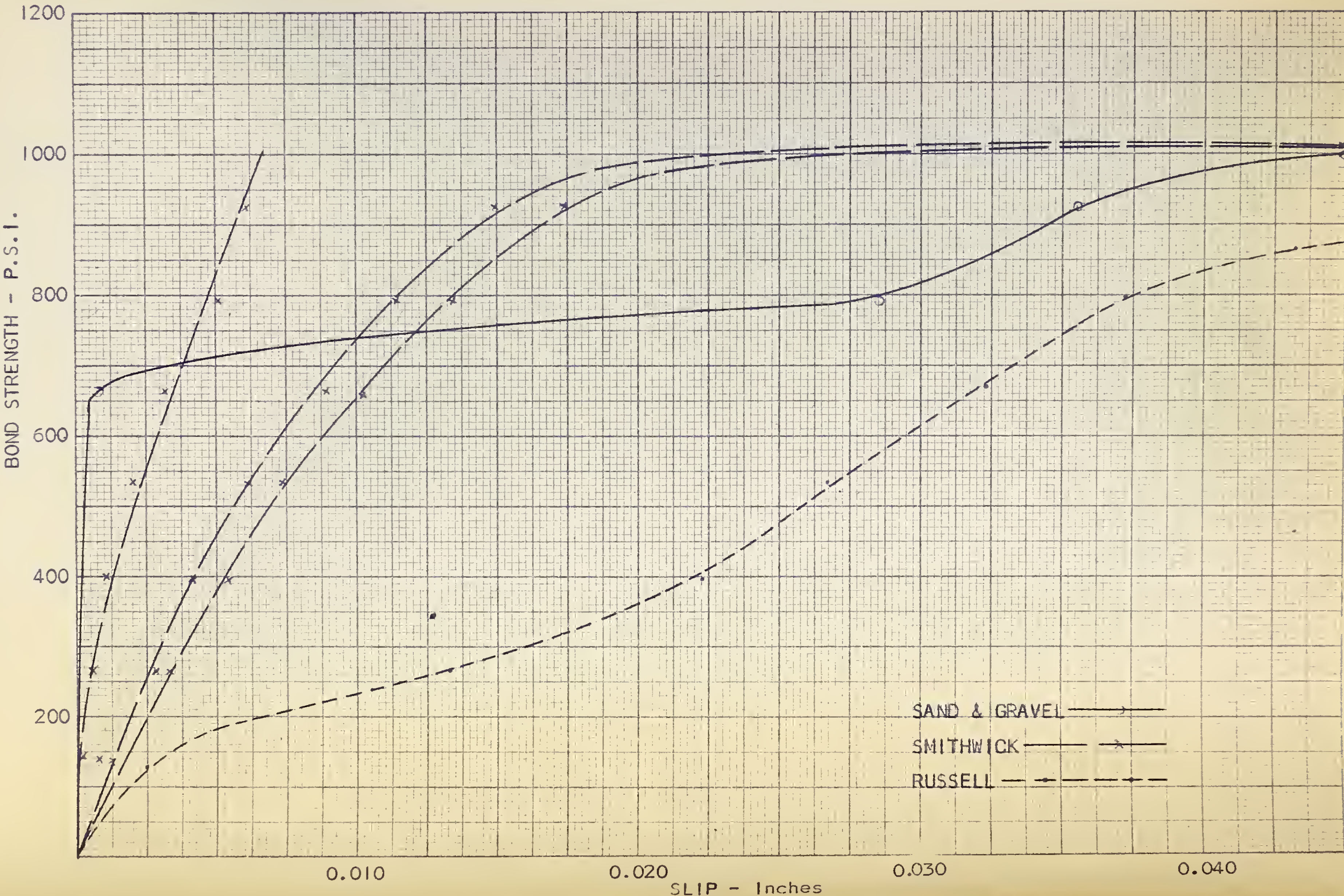


FIG. 40

LOADED END SLIP CURVES - 1" DIA. HIBOND BARS W/C = 0.50

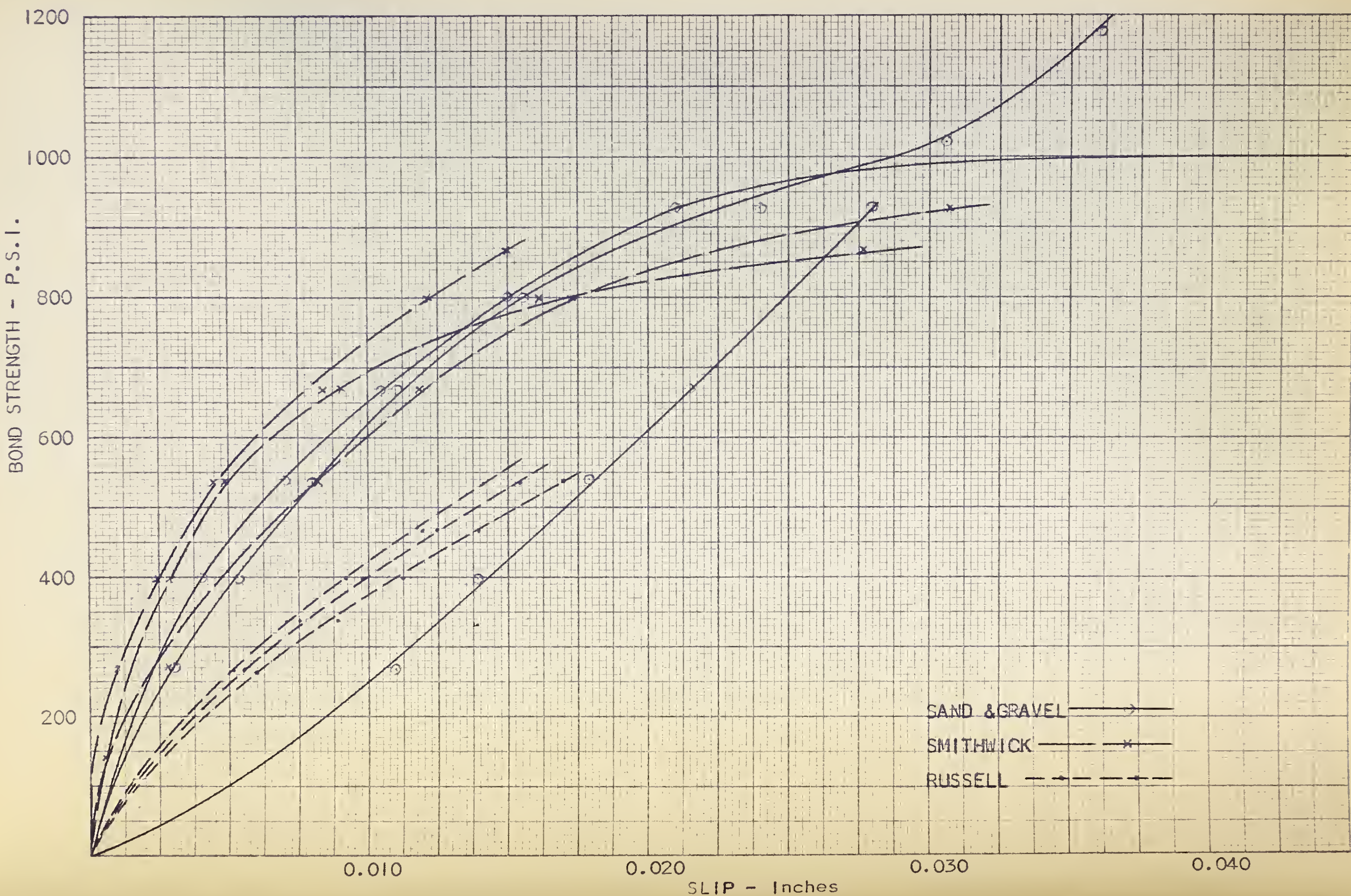


FIG. 41

LOADED END SLIP CURVES - 1" DIA. HIBOND BARS W/C = 0.60

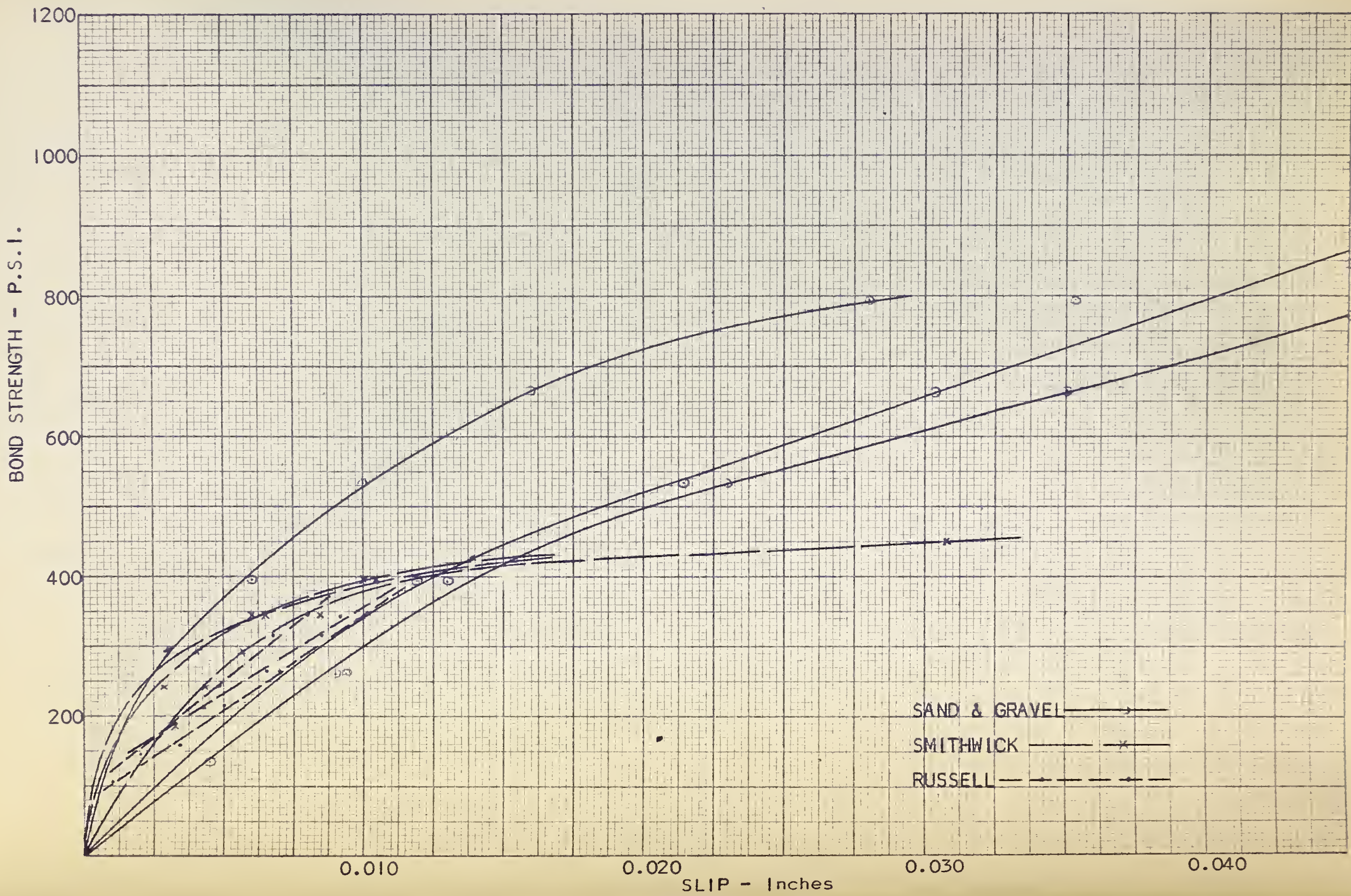


FIG. 42

LOADED END SLIP CURVES - 1" DIA. HIBOND BARS W/C = 0.70

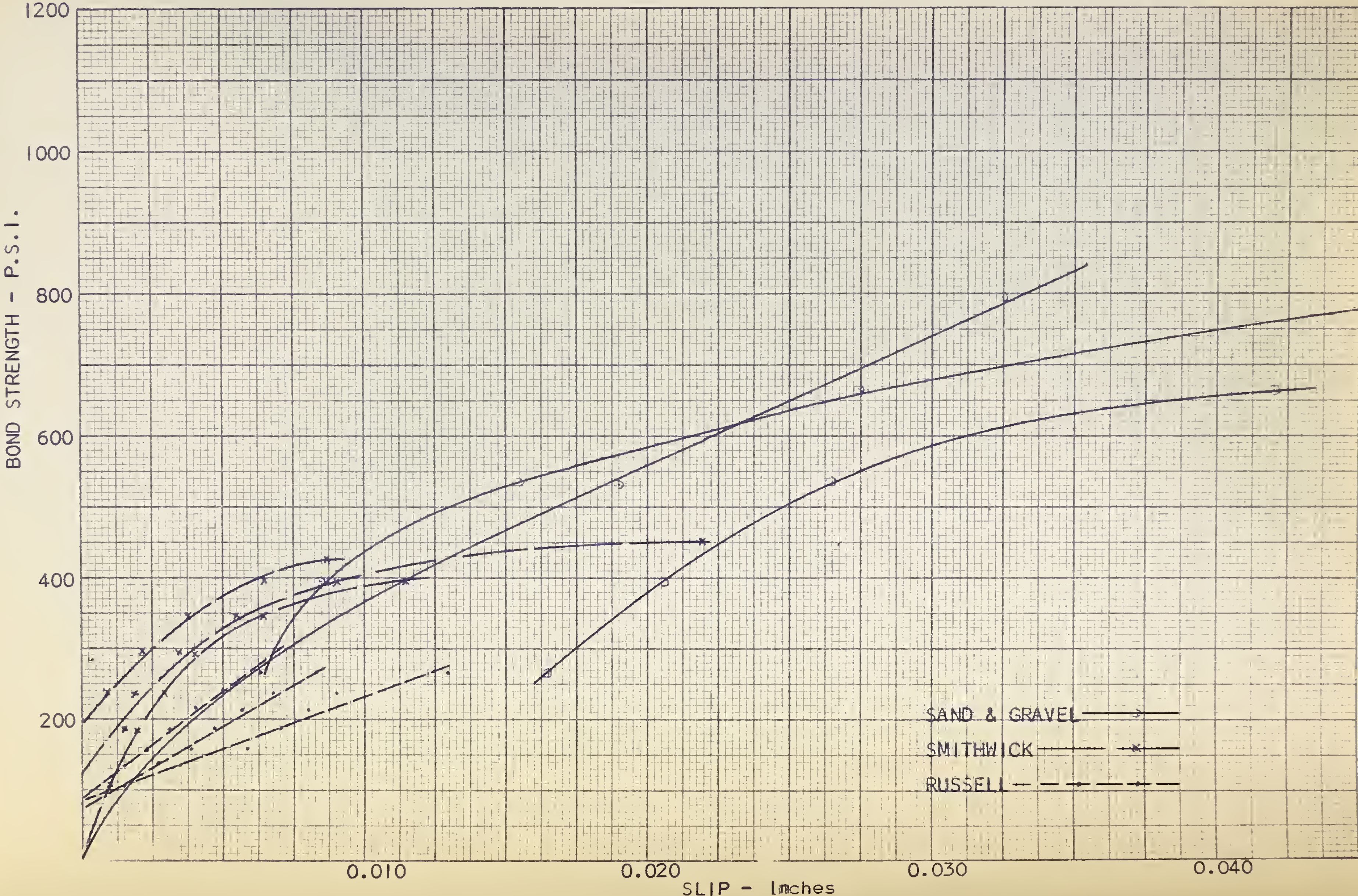


FIG. 43

FREE END SLIP CURVES - 1" DIA. HIBOND BARS W/C = 0.40

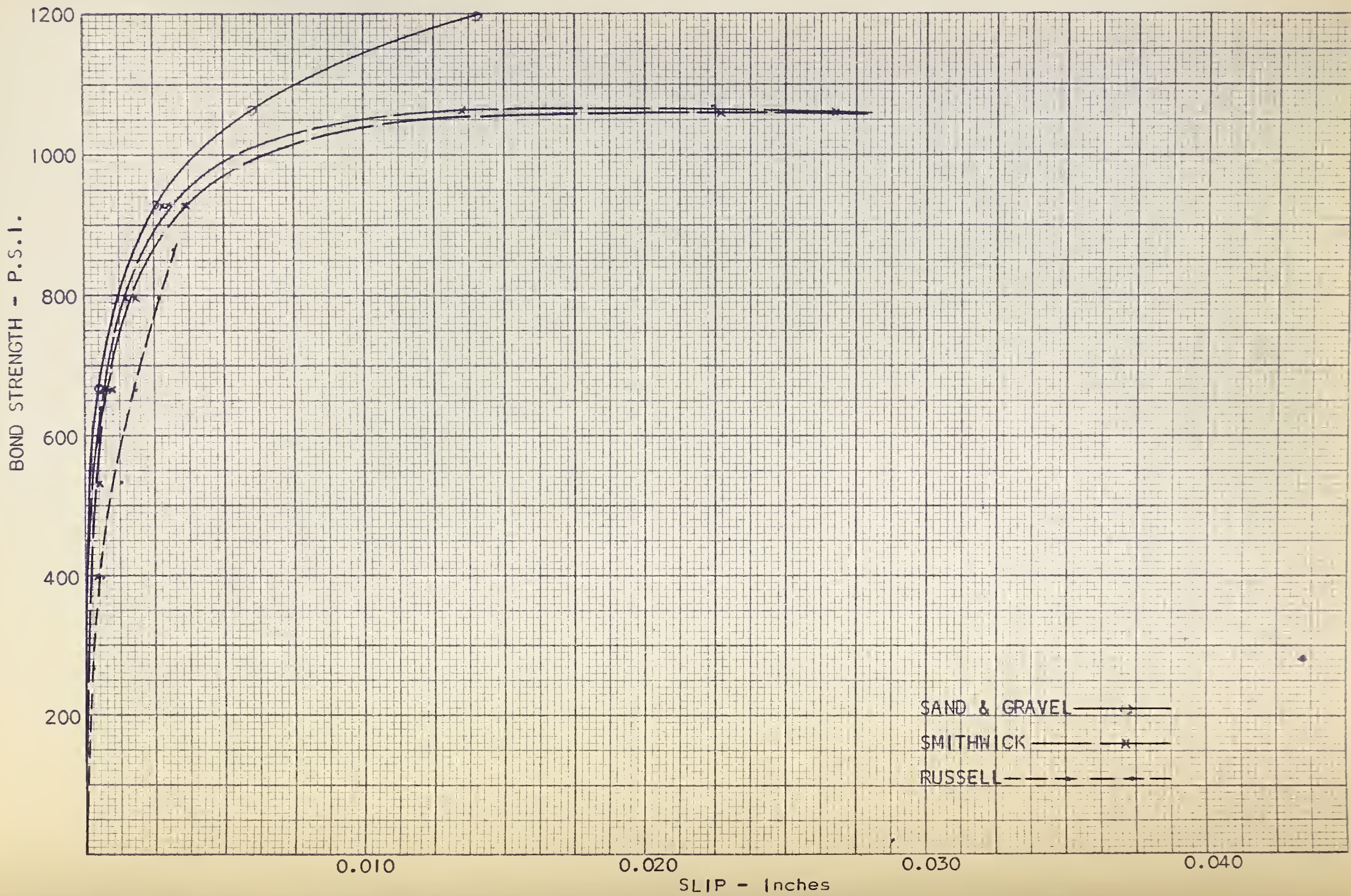


FIG. 44

FREE END SLIP CURVES - 1" DIA. HIBOND BARS W/C = 0.50

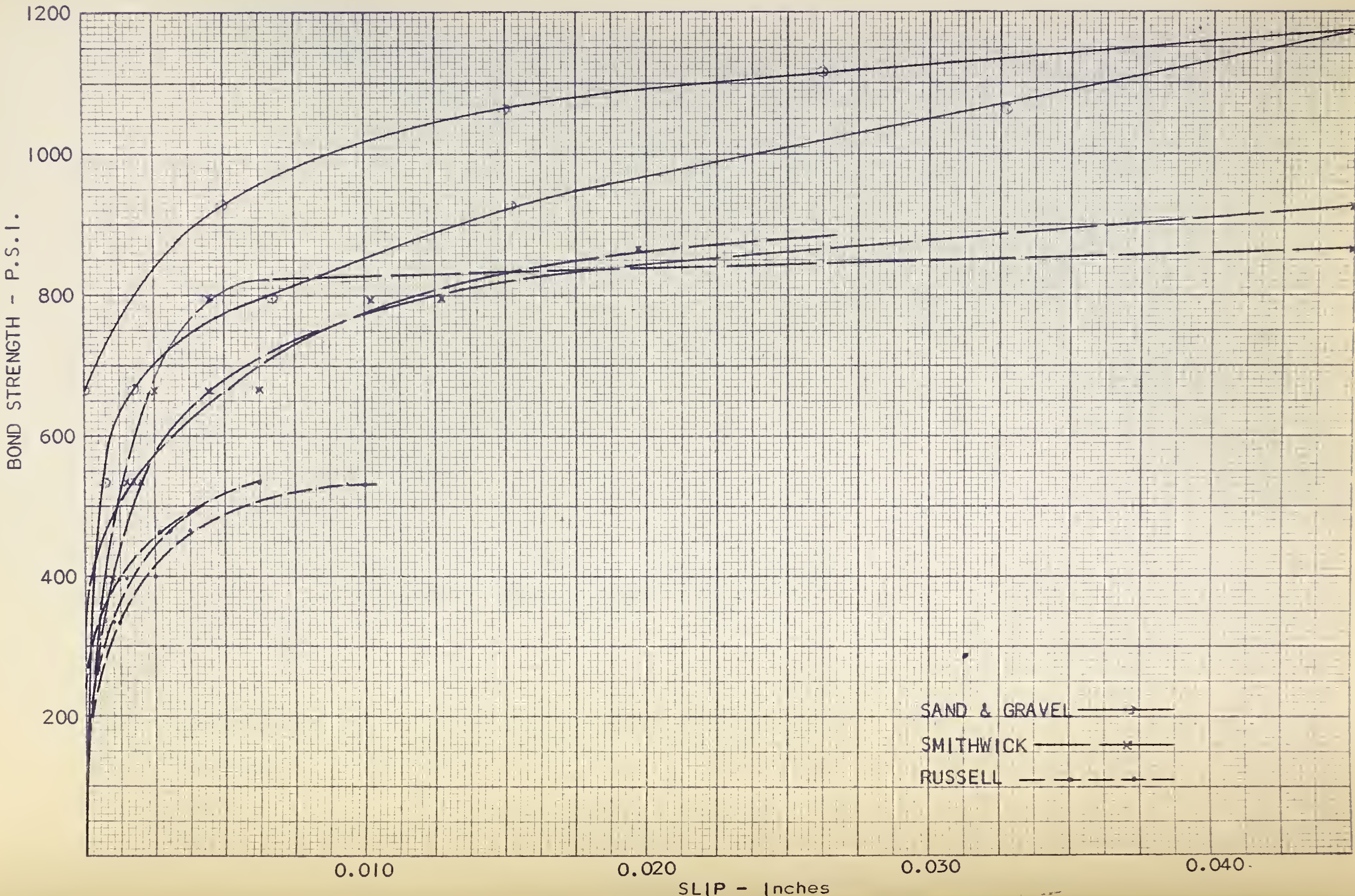


FIG. 45

FREE END SLIP CURVES - 1" DIA. HIBOND BARS W/C = 0.60

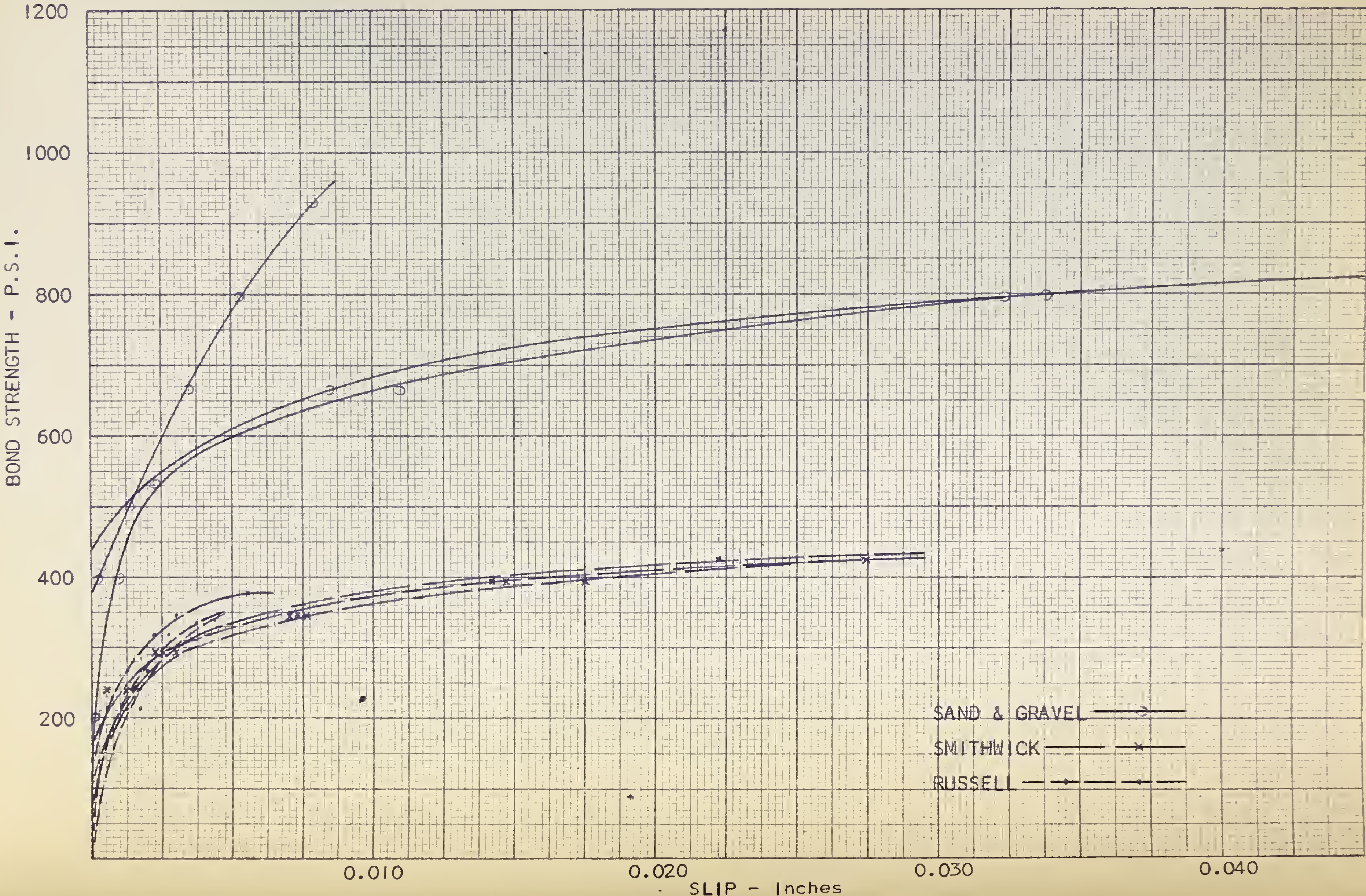


FIG. 36

FREE END SLIP CURVES - 1" DIA. HIBOND BARS W/C = 0.70

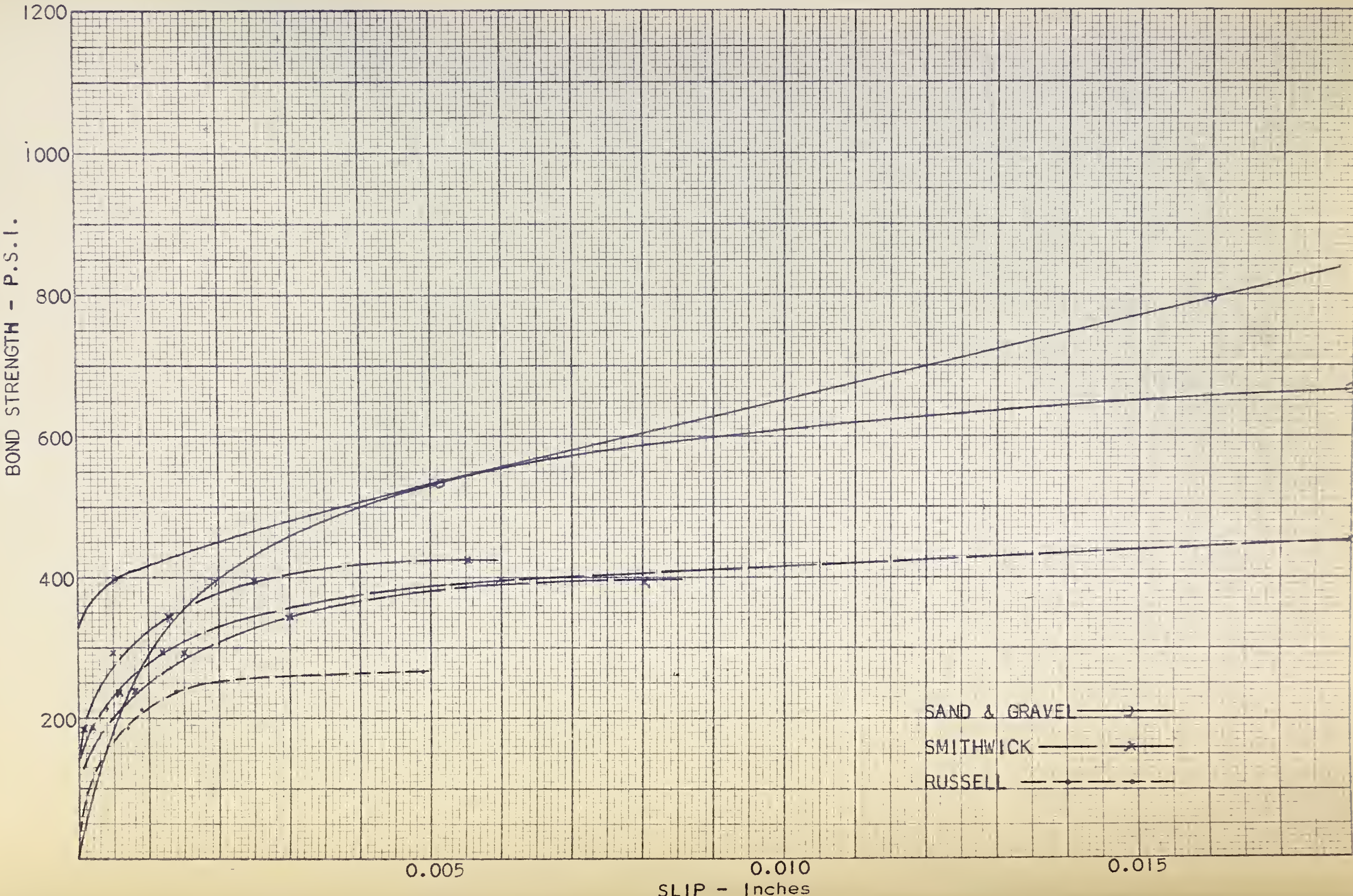


FIG. 47

FREE END SLIP CURVES - 1" DIA. PLAIN BARS W/C = 0.40

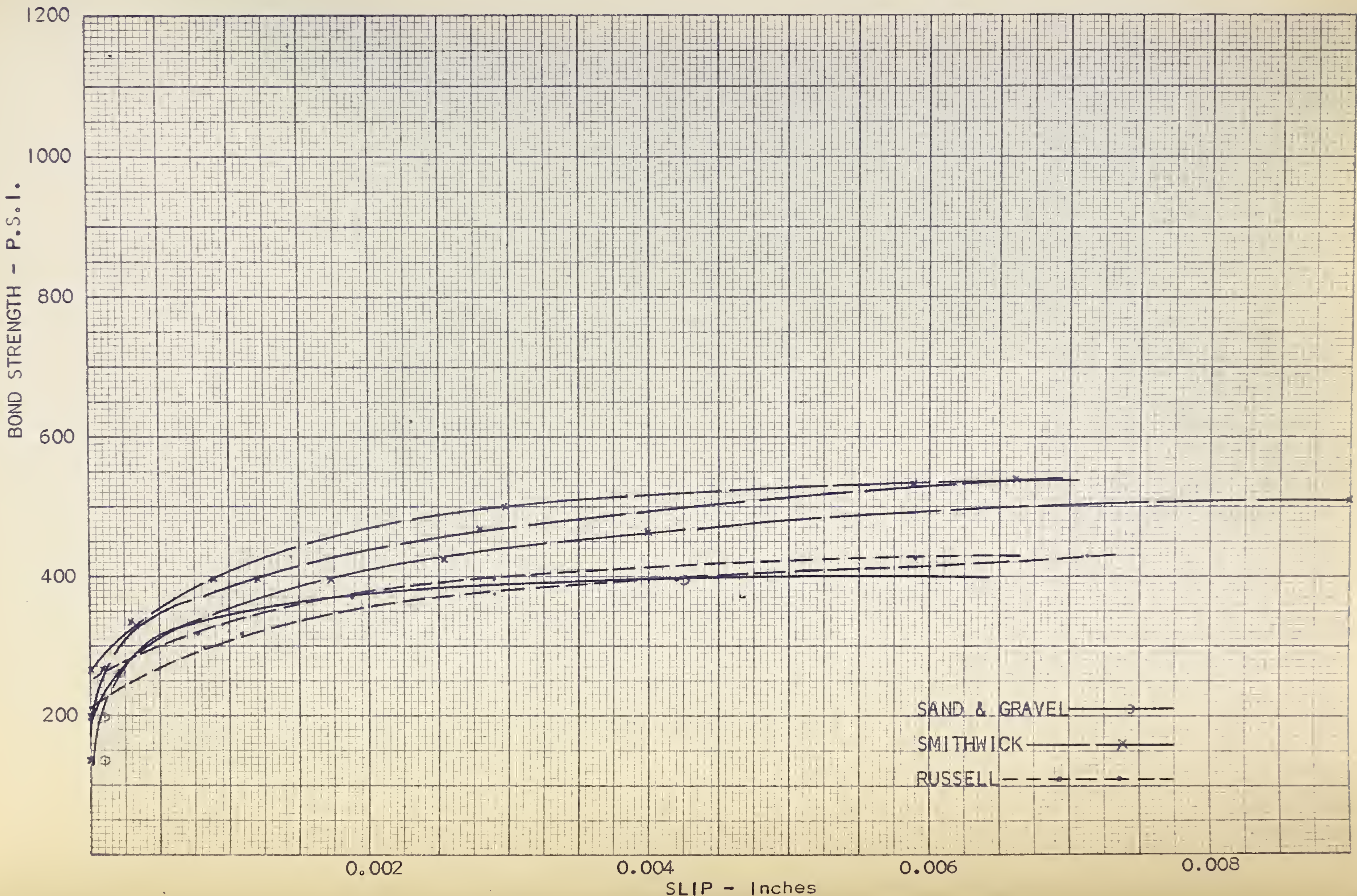


FIG. 48

FREE END SLIP CURVES - 1" DIA. PLAIN BARS W/C = 0.50

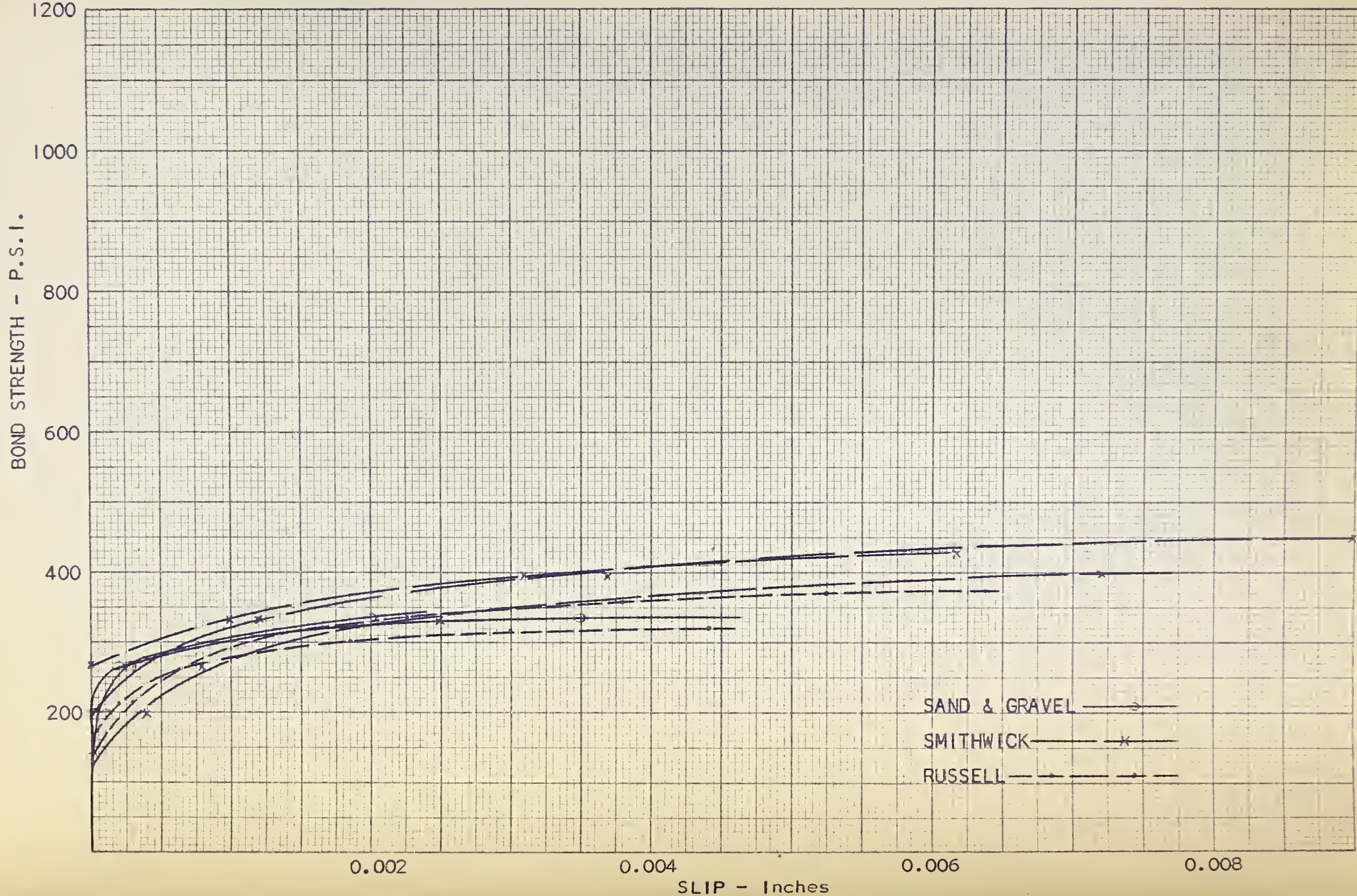


FIG. 49

FREE END SLIP CURVES - 1" DIA. PLAIN BARS W/C 0.60

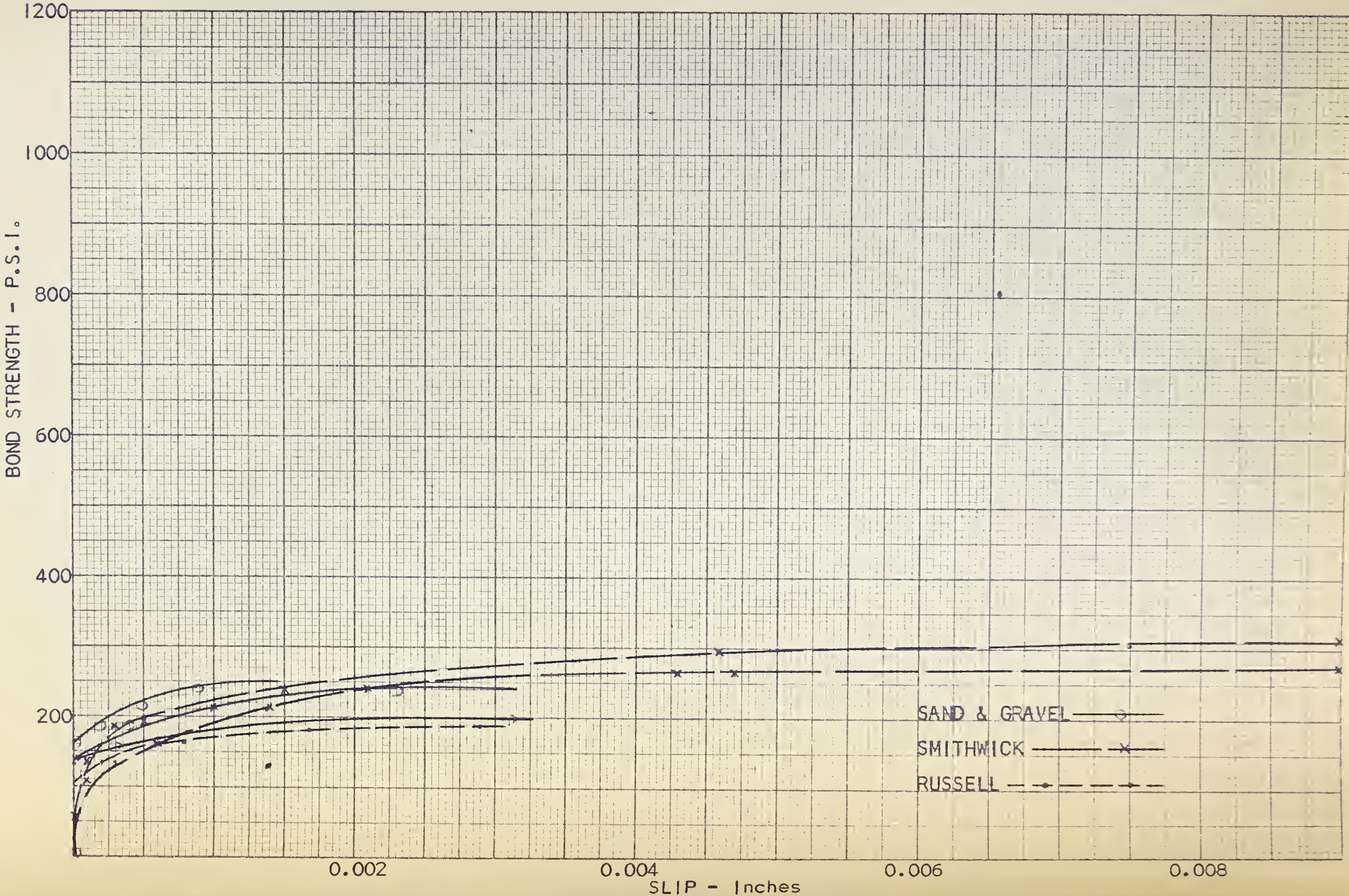


FIG. 50

FREE END SLIP CURVES - 1" DIA. PLAIN BARS W/C = 0.70

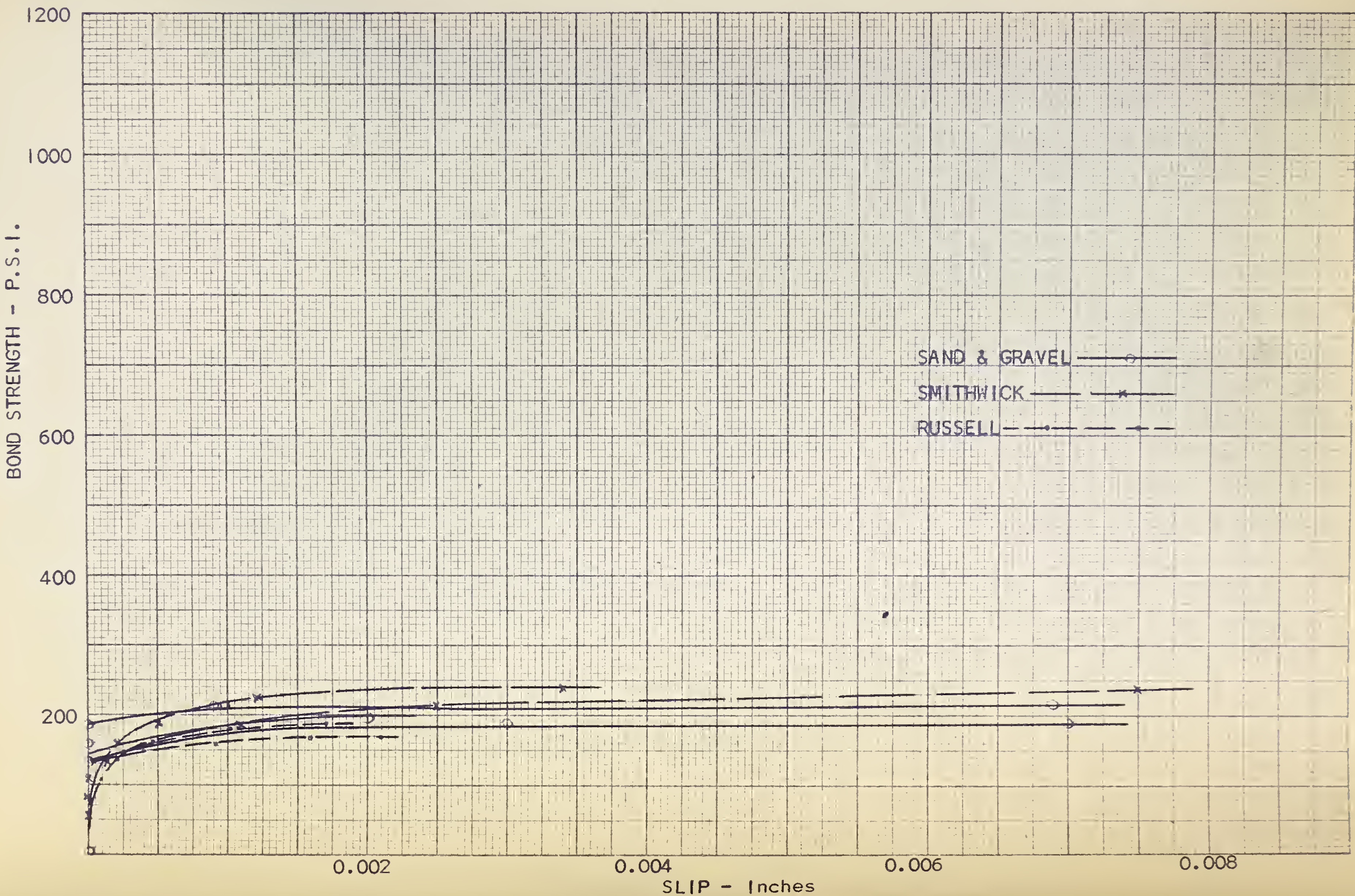


FIG. 51

BOND STRENGTH vs COMPRESSIVE STRENGTH - SAND & GRAVEL HIBOND BARS

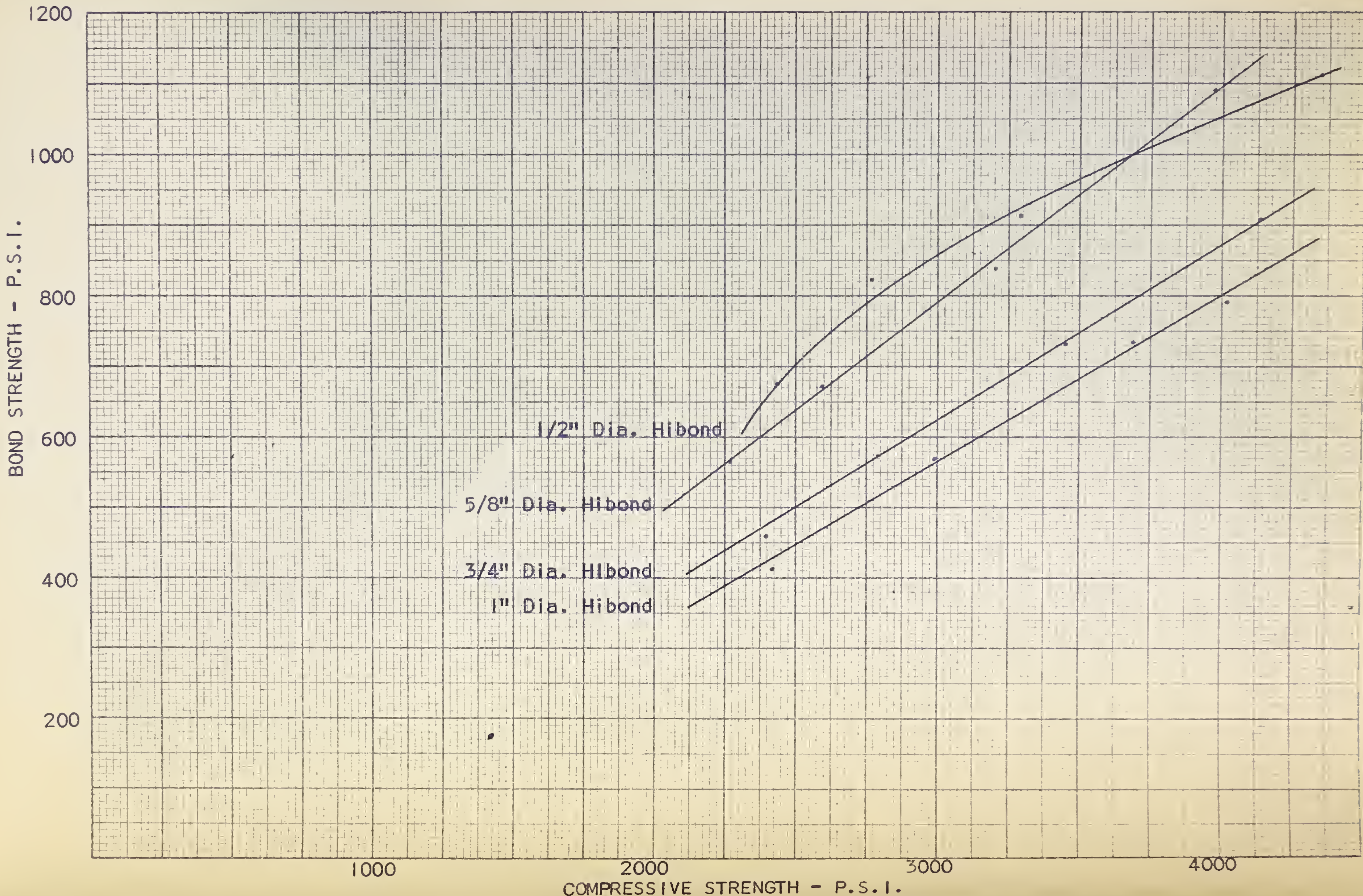


FIG. 52

BOND STRENGTH vs COMPRESSIVE STRENGTH - SMITHWICK AGG. HIBOND BARS

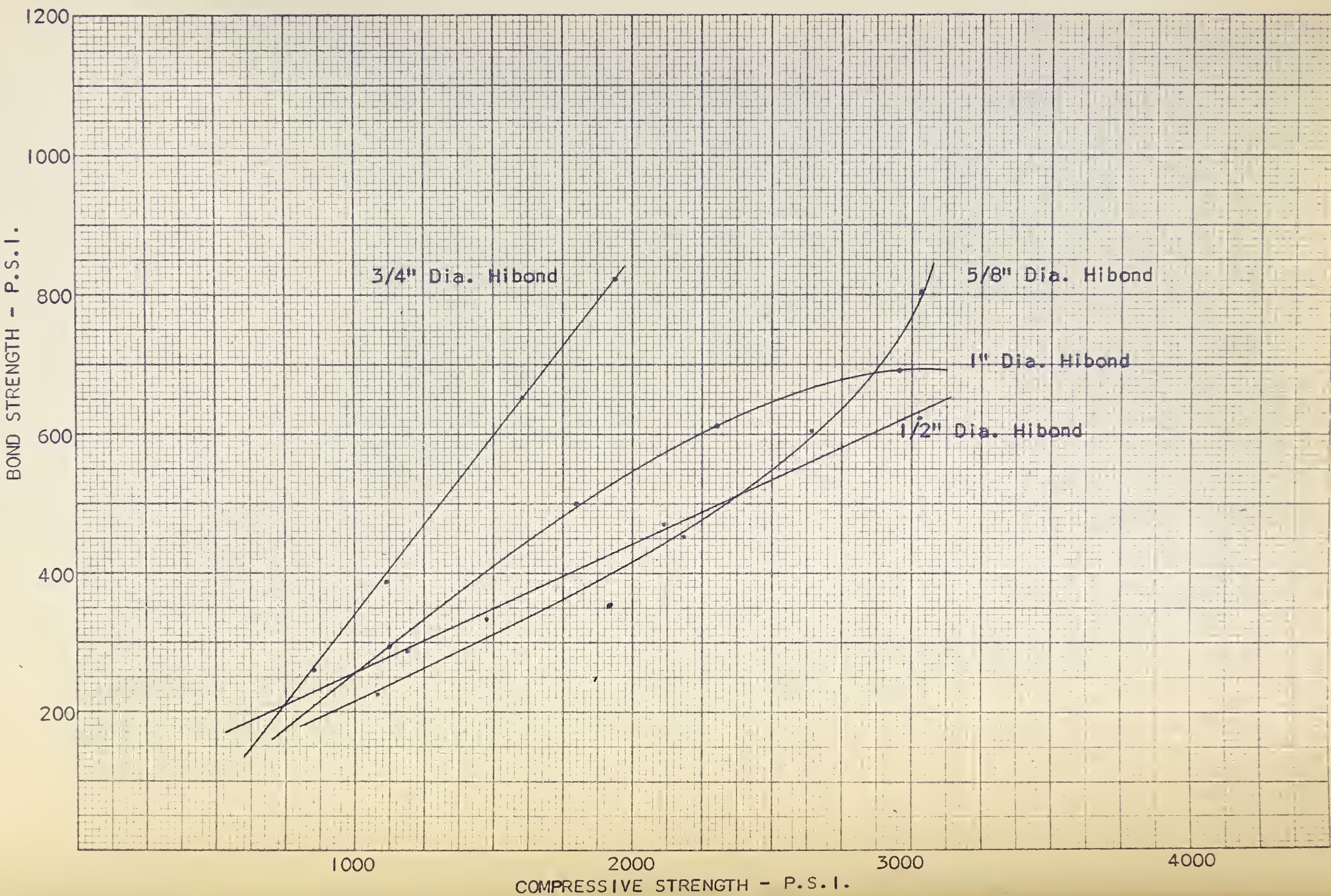


FIG. 53

BOND STRENGTH vs COMPRESSIVE STRENGTH - RUSSELL'S AGG. HIBOND BARS

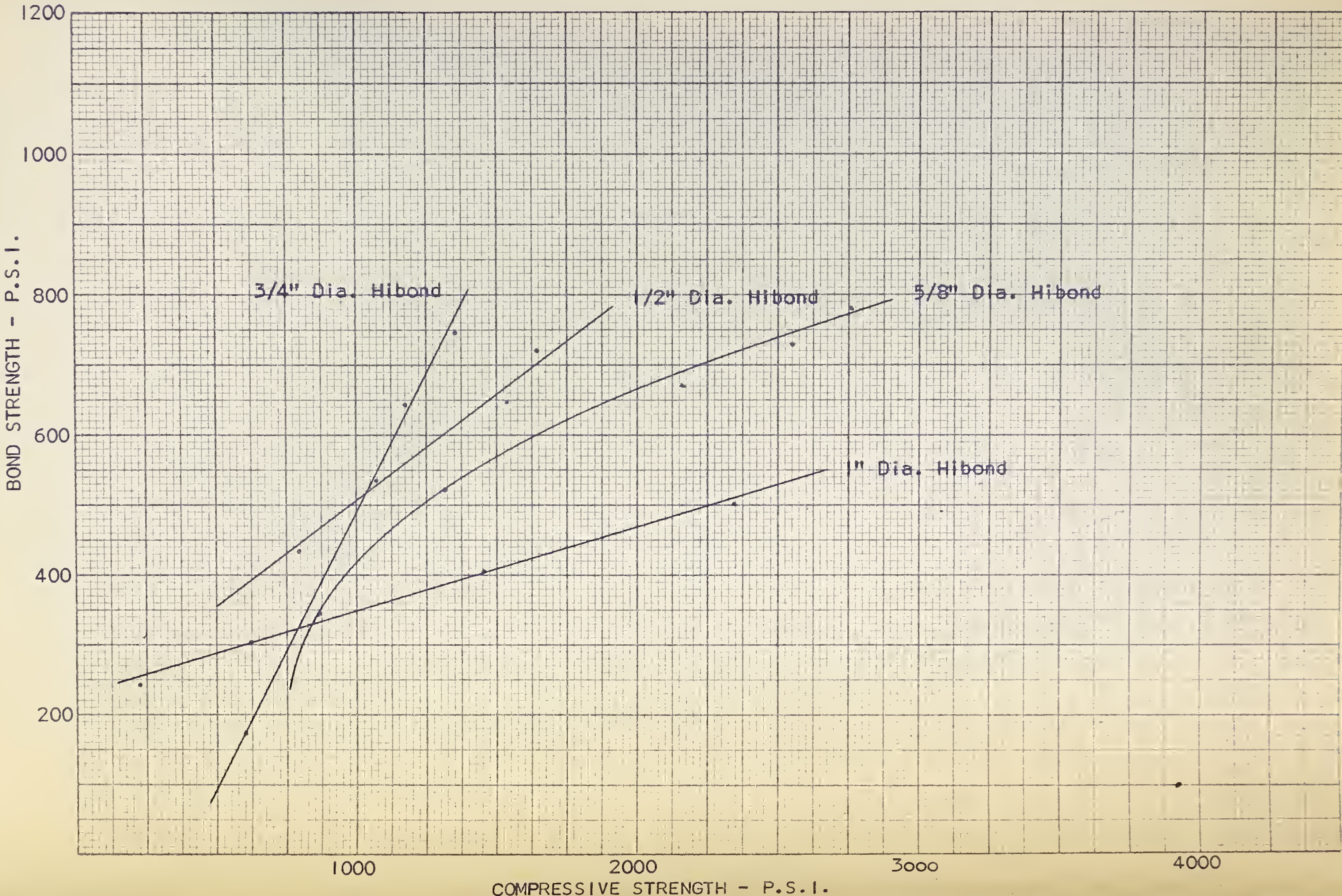


FIG. 54

BOND STRENGTH vs COMPRESSIVE STRENGTH - SAND & GRAVEL PLAIN BARS

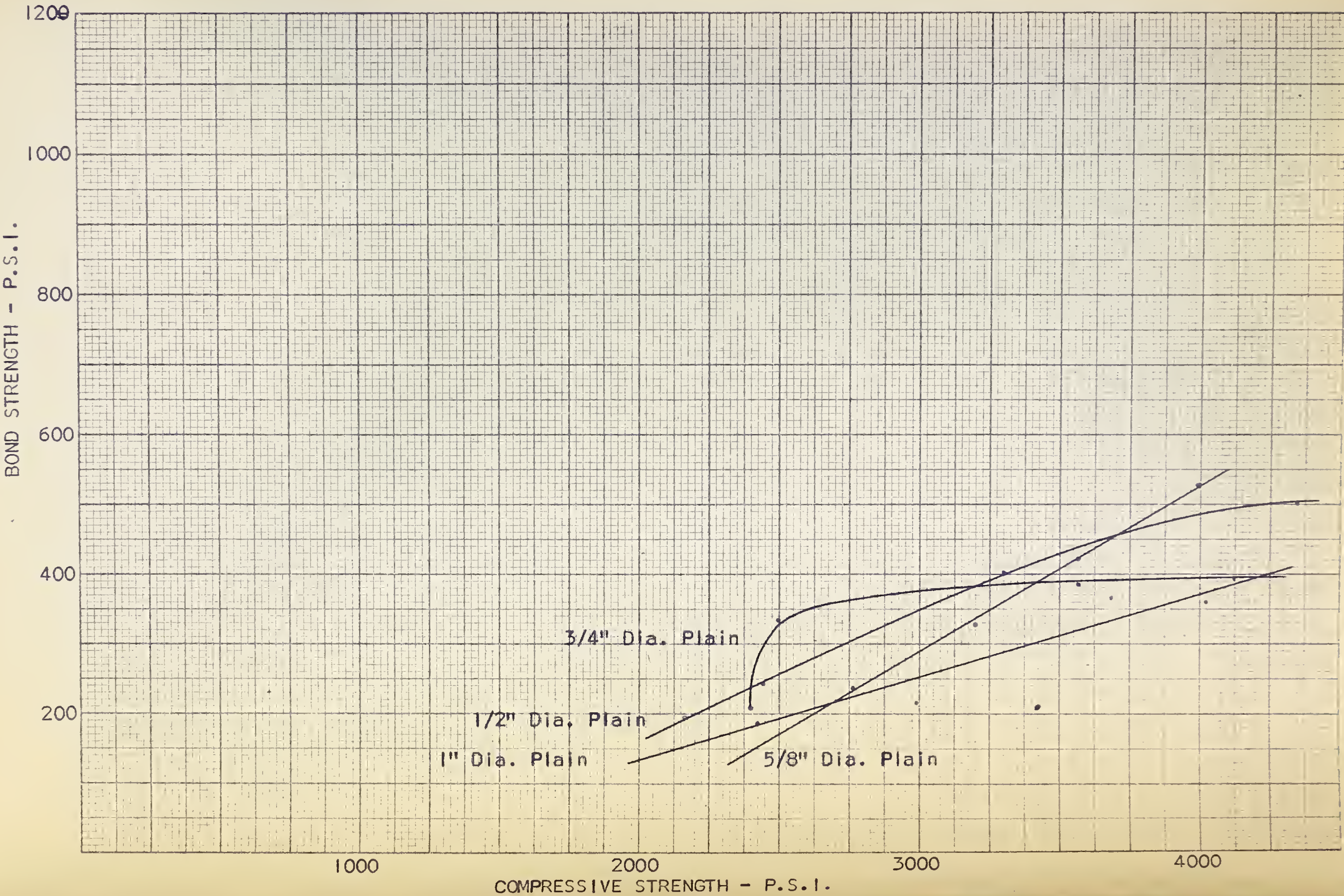


FIG. 55

BOND STRENGTH vs COMPRESSIVE STRENGTH - SMITHWICK AGG. PLAIN BARS

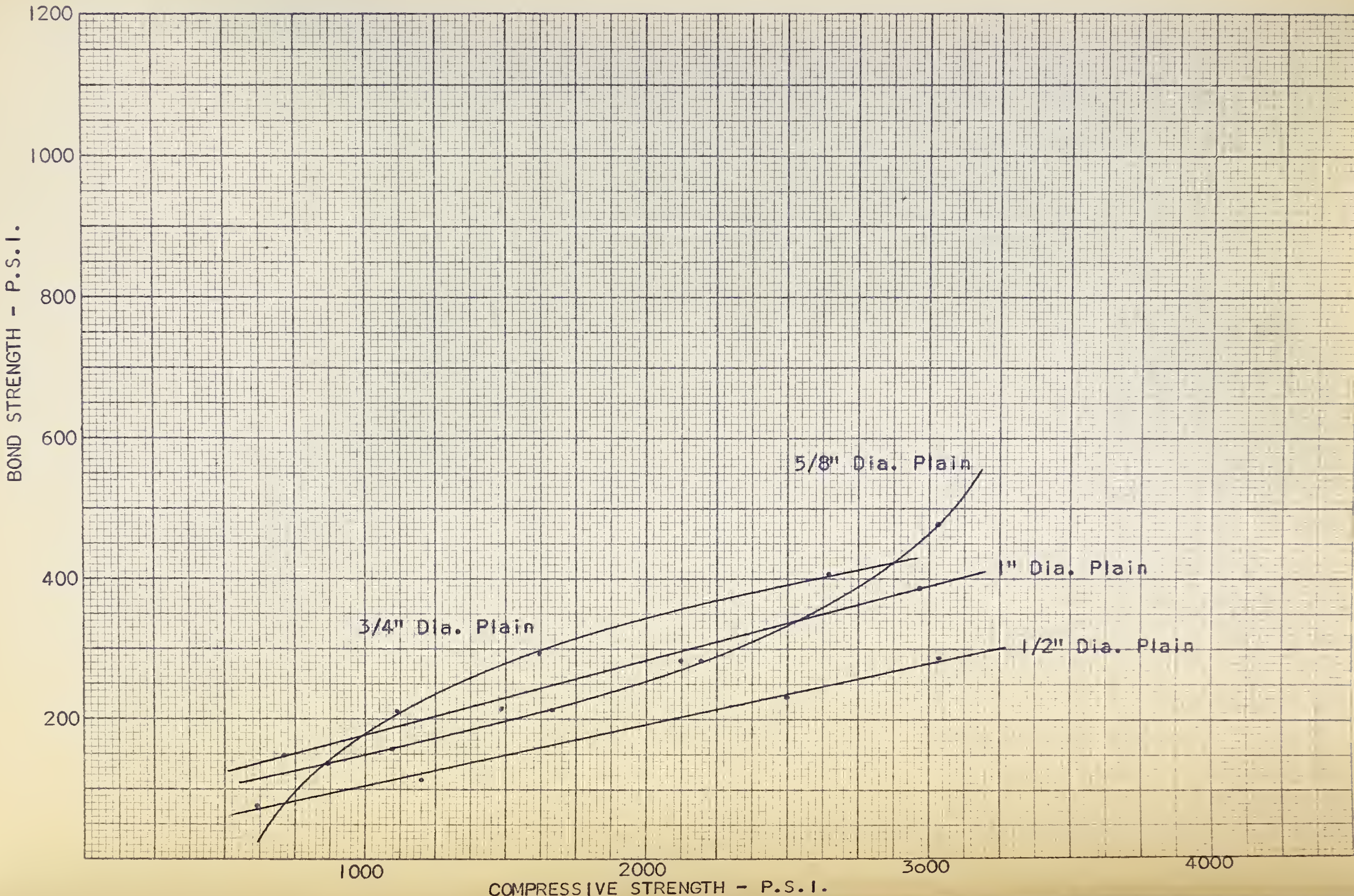
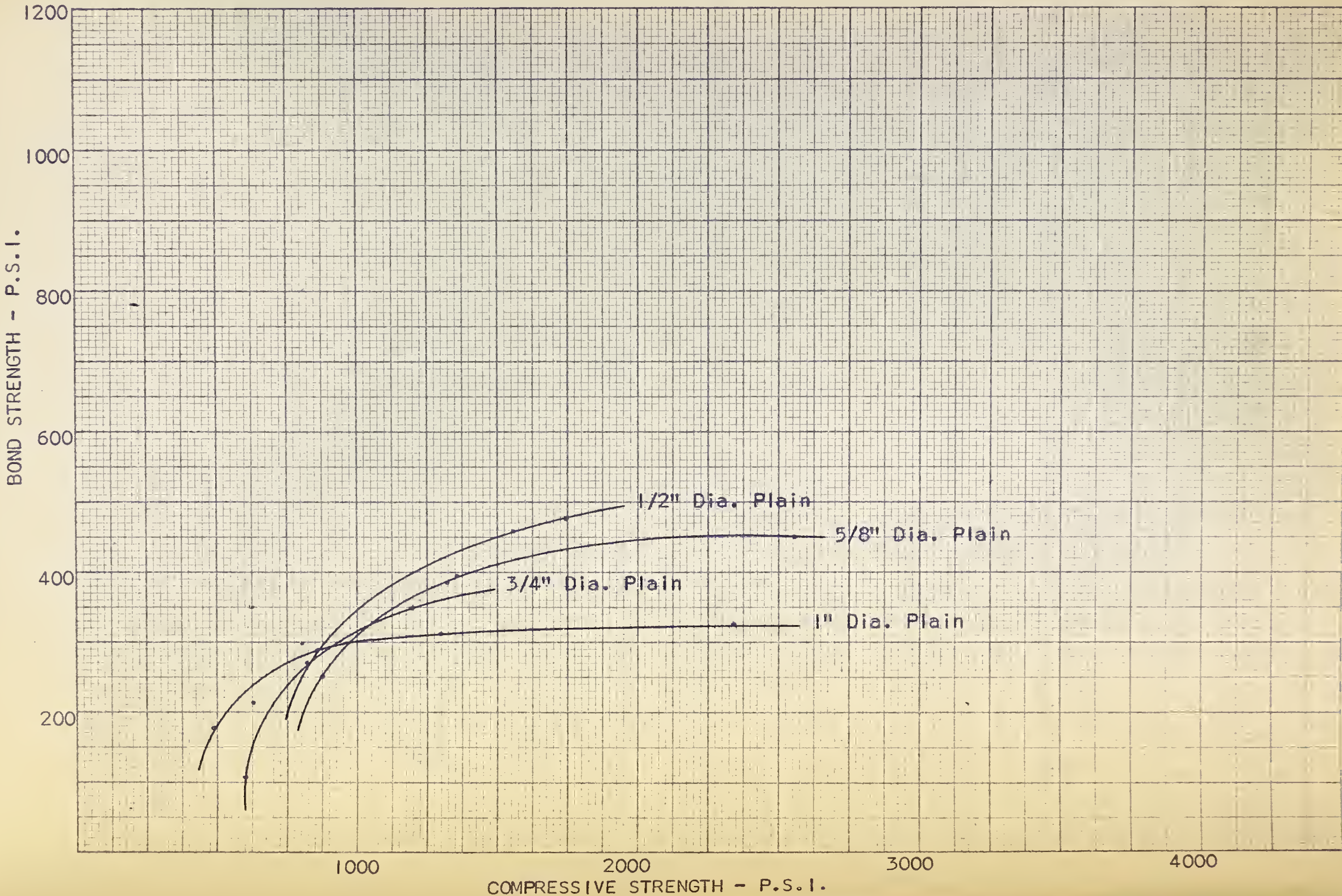


FIG. 56

BOND STRENGTH vs COMPRESSIVE STRENGTH - RUSSELL'S AGG. PLAIN BARS



Chapter VIII

Summary and Conclusion

In summarizing the principal features of the tests described in this thesis, the following conclusions have been formulated:

1) The average unit weights of the fine and coarse light weight aggregates used were respectively 43.0 and 34.2 lbs. per cubic foot by dry loose measure for Russell's aggregate and 45.6 and 36.6 lbs. per cubic foot for the Smithwick's aggregate. Smithwick's aggregate also comes in an intermediate size of which the average unit weight was 43.7 lbs. per cubic foot by dry loose measure.

2) The specific gravities on a bulk basis were, for the fine and coarse light weight aggregates used, 2.08 and 0.98 respectively for Russell's aggregate and 2.15 and 1.23 for Smithwick's aggregate, as well as 1.82 for the intermediate size of Smithwick's aggregate. The tests indicated that the longer the period of soaking, the higher the value obtained for specific gravity. What must be remembered is that the consistency and repeatability of the determination of specific gravity never completely approaches the accuracy obtained from sand and gravel.

3) Concrete was made with light weight aggregates with no particular departure from ordinary methods. The only difference was a pre-soaking of the aggregate to a saturated surface dry condition. In practice, however, this would not be done and hence mixing procedure would be the same. Of the two types of mixers used the paddle type mixer was the more efficient for light weight concrete as it cut segregation to a minimum. Light weight concrete is harsher than sand and gravel concrete necessitating air-entrainment. No great difference was noted in workability of the air-

entrained light weight concrete as compared to the sand and gravel concrete. Finishing was slightly more difficult than for sand and gravel concrete.

4) Slump is not indicative of the consistency of light weight concrete to the same extent as for sand and gravel concrete.

5) Because of the high water absorption by light weight concrete aggregate, the calculation of the water cement ratio involves an accurate determination of the amount of absorbed moisture. The tests indicated that the absorption allowance for a light weight aggregate is not a fixed quantity, but depends upon the time of soaking and initial absorption of the aggregate. The absorption of the light weight aggregates and the consequent allowances used for concrete mixes were 18.3% and 17.3% for Russell's fine and coarse aggregates and 15.7% and 14.1% for Smithwick's fine and coarse sizes. Smithwick's intermediate aggregate had an absorption of 18.2%. All absorption figures are on a 24 hour basis.

6) Smithwick's aggregate yielded strengths just slightly lower than those of sand and gravel. Russell's aggregate yielded considerably lower results but it was felt that this was due to extremely poor gradation of the fines. Comparable strengths using a well graded light weight aggregate could be obtained by using the same cement factors as for sand and gravel concrete.

7) In designing light weight concrete a cement factor should be used and not a w/c ratio. Because of the high absorption quality of light weight aggregate the w/c ratio is of no significance as there is no means of evaluating the absorption taking place in the mix. The w/c ratio law, of course, still holds but is not applicable in light weight concrete design. Therefore design curves should be based on cement contents for light weight

concrete and not on w/c ratio curves. There is no indication that the strength of any of the mixes was limited by the strength of the aggregate particles.

8) The durability of light weight concrete under freezing and thawing conditions is far superior to that of sand and gravel concrete where air entrainment is not used. The light weight concrete exhibited durability factors based on the elastic modulus in a range of 64 to 94 while comparable sand and gravel counterparts exhibited a durability factor of zero. Air-entrainment imparted excellent durability properties to the light weight and the sand and gravel concrete, although it was necessary only in the latter. Russell's aggregate (expanded clay) exhibited the best durability factors.

9) The ratio of bond resistance to compressive strength at the age of 28 days was essentially the same for like mixes of light weight and sand and gravel concrete. If anything, the light weight concrete exhibited slightly greater bond resistance for equivalent compressive strengths. The results indicate that the same design rules for bond can be applied to the light weight concrete as for the sand and gravel concrete.

10) A series of tests not included in this report indicate that a low modulus of elasticity is one of the outstanding characteristics of light weight concrete. Over the comparatively wide range of compressive strengths tested the values of initial modulus of elasticity for light weight concrete are about 55% of the values of sand and gravel concretes of corresponding strength. Air entrainment further lowered the elastic modulus. However, corresponding decreases were also noticed for the sand and gravel concrete.

Chapter IX

Recommendations

In connection with the tests which have been run, numerous occasions occurred when it was apparent that the test was not being carried far enough, or the testing procedure was not adequate to determine the best possible results. Herewith are presented a number of recommendations for anyone who should continue these tests or enlarge upon any specific phase.

The first difficulty encountered was in the mix design. Under the circumstances the best possible method was used. However, as the tests progressed it became apparent that the use of a w/c ratio in connection with light weight concrete design was not practical. The w/c ratio law still holds but due to absorption is almost impossible to control. The recommendation is that light weight mixes should be proportioned by volume initially and where weight is needed it could be obtained by using the dry unit weights. Using a volume batch method together with a cement content curve would give the necessary data for mix design.

The pull-out tests were not adequate to give good bond values. Because of more emphasis being placed upon the load-slip curves precautions would have to be taken to ensure that the bars are perpendicular to the concrete surface. A series of beam bond tests in connection with the pull-out tests would correlate the relation between beam bond and pull-out tests. The many factors which can enter into bond determination such as settlement, method of casting, position during casting, etc., should be considered in connection with any detailed bond stress determination.

Hognestad has presented the effect of air-entrainment on bond for sand

and gravel concrete. This same determination would be more necessary in the light weight field as light weight concrete requires air-entrainment to give it workability.

In connection with further studies two important properties to determine would be shrinkage and creep.

The actual dirth of published material on light weight concrete becomes very apparent when one tries to find design data and coefficients. The field for further research in light weight concrete is unlimited as engineers and architects require much more information than is now available.

Bibliography

1. Tests of Plain and Reinforced Concrete Made with Haydite Aggregates. Richart & Jensen. Bull. #237. Engineering Experimental Station, U. of Illinois. Oct. 27, 1931, p. 17.

2. The Report of an Investigation on the Condition and Physical Properties of Expanded Shale Reinforced Concrete after Thirty-Four Years' Exposure to Sea Water. Technical Problems Committee, Expanded Shale Institute, Cedric Wilson, Chairman.

3. Field Practice in Lightweight Concrete. John A. Murlin and Cedric Wilson. A.C.I. Journal. Sept. 1952.

4. Lightweight-Aggregate Concrete. Kluge, Sparks & Tuma. A.C.I. Journal, May 1949.

5. Burned Shale and Expanded Slag Concrete with and without Air-entraining Admixture. P. H. Petersen. A. C. I. Journal, Oct. 1948.

6. An Investigation of the Air Entrainment in Concrete and Its Effect on Durability. K. R. Lauer, Masters' Thesis 1948, University of Alberta.

7. An Investigation of the Effects of Air-Entraining Agents, Various Cements, and Curing Conditions on Manufacture of Concrete Specimens. J. L. Jaspar, Masters' Thesis 1950, University of Alberta.

8. Production of Lightweight Concrete Aggregate from Clays, Shales, Slates, and other Materials. Conley, Hewett Wilson, Klinefilter. United States Department of Interior, Bureau of Mines. R. I. 4401. Nov. 1948.

9. A.S.T.M. Standards on Mineral Aggregates, Concrete, and Non-bituminous Highway Materials.

10. C.S.A. Standards for Cement.

11. Production of Lightweight Aggregates. H. G. Iverson. Rock Products. Feb. 1954.

12. Evaluation of Air-Entraining Admixtures for Concrete. Jackson & Timms, Bureau of Public Roads. Public Roads Journal. Feb. 1954.

13. Automatic Accelerated Freezing and Thawing Apparatus for Concrete. Charles E. Wuerpel & Herbert K. Cook. Proc. A.S.T.M. Vol. 45. 1945.

14. Variation on Density of Lightweight Concrete Aggregate. Harold S. Sweet. A.S.T.M. Bull. Sept. 1952. No. 184.

15. A Proposed Standard Deformed Bar for Reinforcing Concrete. Carl A. Menzel. Concrete Reinforcing Steel Institute Semi-Annual Meeting. 1941.

16. Bond of Concrete Reinforcing Bars. Arthur P. Clark. A.C.I. Journal. Nov. 1949.

17. Effect of Entrained Air on Bond Between Concrete and Reinforcing Steel. E. Hognestad & C. P. Seiss. A.C.I. Journal, Vol. 21, #8, Apr. 1950.

18. Studies of Concrete Containing Entrained Air. S. Walker & D. L. Bloem. J.A.C.I. Vol. 17, No. 6. June 1946.

19. Function of Entrained Air in Concrete. H. L. Kennedy. Proc. J.A.C.I. Vol. 39. 1943.

20. Correlation Between Laboratory Accelerated Freezing and Thawing and Weathering at Trent Island, Maine. T. B. Kennedy & Katherine Mather. A.C.I. Journal, Vol. 50, #2. 1953.

21. A.S.T.M. Symposium on Mineral Aggregates. S.T.P. 83. Oct. 1948.

22. Review of Research in Ultimate Strength of Reinforced Concrete Members. C.P. Seiss. A.C.I. Journal, Vol. 23, No. 10, June 1952.

23. Some Factors Influencing the Results of Pull-Out Bond Tests. Carl A. Menzel.

APPENDIX I

LABORATORY EQUIPMENT

Appendix I

Laboratory Equipment

Laboratory Concrete Mixers

Two types of concrete mixers were used in connection with this investigation:

(1) "Lancaster" Mixer

(2) $3\frac{1}{2}$ S – End Discharge Mixer

(1) Lancaster Mixer

Description

The "Lancaster" Mixer used is a type S K G (see photo) manufactured by the Volta Manufacturing Company Limited, Welland, Ontario. This mixer is designed only for laboratory work. It has an open circular removable drum sitting in an horizontal position with vertical mixing blades which rotate in an opposite direction to the drum. The vertical blades can be raised to allow removal of the drum or to facilitate better removal of the mixed concrete. The mixer is powered by a three horsepower Wagner Electric motor and is operated at a speed of 33 revolutions per minute. The maximum capacity of the mixer is two cubic feet but better mixing takes place when this is reduced to a working capacity of about $1\frac{1}{2}$ cubic feet.

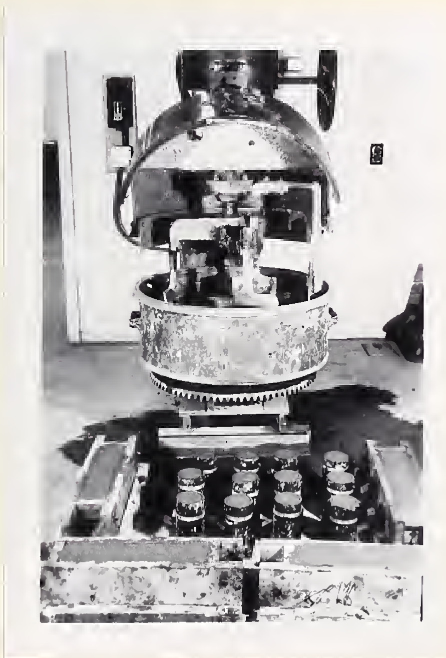
Mixing Procedure

In using the Lancaster mixer the operating instructions suggest that the dry materials be added first and then the water. In all cases the fine and coarse aggregates were put in first and mixed for one-half minute. The cement was then added and also allowed to mix for one-half minute. This was followed by the water and a minimum mixing time of two minutes. When an air-entraining

agent was used it was added to the fine aggregate to insure proper distribution throughout the mix. This mixer was used to pour all the strength cylinders and freeze-thaw beams.

Efficiency of Mixing

The "Lancaster" Mixer gave a uniformly mixed concrete in all cases. There was at no time apparent segregation which seemed due to the mixer. There was, however, a small area about the bottom inside perimeter in which the fine aggregate could be untouched during the mixing operation. As a result, despite the fact that the quantity was very small, precautions were taken not to get this unmixed portion into the cylinders or beams.



Photograph No. 25 - Lancaster Mixer with a typical pour in front.

(2) $3\frac{1}{2}$ S – End Discharge Mixer

Description

The $3\frac{1}{2}$ S – End Discharge concrete mixer (see photo) is a tilting drum type mixer manufactured by the Kwik-Mix Company, Port Washington, Wisconsin. This mixer is designed primarily as a field mixer and is of the general type used by small contractors. It has an open end tilting drum which facilitates easy removal of the concrete. The mixer is powered by a $1\frac{1}{2}$ horsepower Wagner induction electric motor and is operated at a drum speed of 24 revolutions per minute. Capacity of the mixer is $3\frac{1}{2}$ cubic feet. No difficulty whatever is experienced when operating the mixer at this capacity.

Mixing Procedure

In using the $3\frac{1}{2}$ S mixer two mixing procedures were used:

(a) In the case of sand and gravel concrete, about three-quarters of the water was put in followed by the coarse aggregate. The sand was then added followed by the cement. This is recommended procedure [1] for sand and gravel concrete mixing.

(b) In the case of light weight concrete the fine and coarse aggregates were put in the mixer first followed by the cement and then the water.

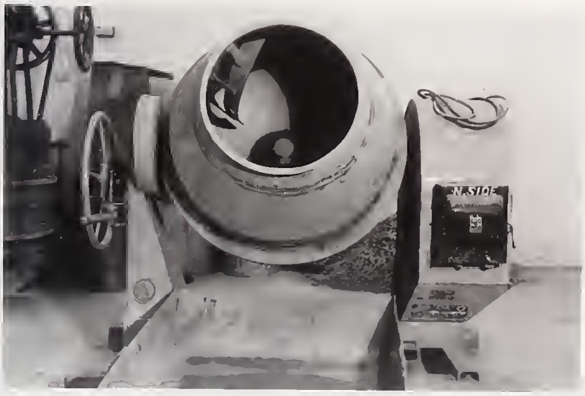
When an air-entraining agent was used it was added as the third material – that is, following the coarse aggregate in sand-gravel mixes, and following the fine and coarse aggregates in the case of light-weight mixes.

The same mixing time sequence was used in connection with this mixer as with the "Lancaster" mixer. This mixer was used to pour all the pull-out test specimens because of its larger capacity.

Efficiency of Mixing

The $3\frac{1}{2}$ S mixer gave a very uniformly mixed sand and gravel concrete but left something to be desired in the case of light-weight concrete. There were definite signs of segregation in the mixer in connection with the light-weight concrete.

The mixer was designed to operate when in the charge position. With the first mix, however, it was found that the tilt in this position was not severe enough for the drum speed. As a result the mixer was operated on the discharge side with the tilt as severe or flat as possible without discharge of the material during the mixing operation.



Photograph No. 26 – $3\frac{1}{2}$ S Mixer.

APPENDIX II

FREEZING AND THAWING APPARATUS

Appendix II

Freezing and Thawing Apparatus

Introduction

The resistance concrete possesses to frost action is dependent upon countless variables each potentially capable of increasing or decreasing this resistance. The more notable of these variables are:

(1) Chemistry of the cement

(2) Degree and rate of hydration

(3) Nature, grading and thermal properties of the aggregates

(4) Homogeneity of the mixture

(5) Size and shape of specimen

(6) Density of mix

(7) Relative permeability of mix

(8) Degree of air entrainment, if any

To try and reproduce the actual weathering that a structure might undergo would be a very difficult and very time-consuming task although some tests are being run in this manner.[20] As a result, to give some indication of relative durability or resistance to frost action and accelerated freeze-thaw, apparatus was set up by Charles E. Wuerpel and Herbert E. Cook[13] and similarly a freeze-thaw unit of this type was set up at the University of Alberta.[6,7] In an accelerated freeze-thaw unit the cycle is speeded up and made more severe; as a result, it gives a reproducible result and a consequent durability factor which is indicative of the resistance of the concrete in question.

Description of Equipment

For a description of the accelerated freeze-thaw apparatus used in connection with the tests carried out during this investigation see page 2, Chapter II, "A Preliminary Study of Effects of Air-Entraining Agents, Various Cements, and Curing Conditions on Manufacture of Concrete Specimen" by J. L. Jaspar 1950 and "An Investigation of the Air Entrainment in Concrete and its Effect on Durability" by Lauer 1948.(6) Some changes were made in the capacity of the unit and as a result in the general dimensions of the hot and cold tanks. The capacity of these tanks was increased by half again as much. The dimensions of the tanks are now:

(1) Cold tank - 36" x 24" x 48"

(2) Hot tank - 36" dia. x 48"

(3) Specimen tank - 24" x 24" x 28"

Because of the added capacity of the cold tank, two more refrigerating plates were added. Outside of this no changes were made in the apparatus except to install two hydromotor automatic valves in place of the check valves formerly used in the lines.

Temperature Records

Figure 58 represents actual temperature records taken at random during the operation of the unit. A concrete beam was made up with a thermocouple in its centre to obtain temperatures actually attained in the centre of the beams. The position of the thermocouples are noted on the figures.

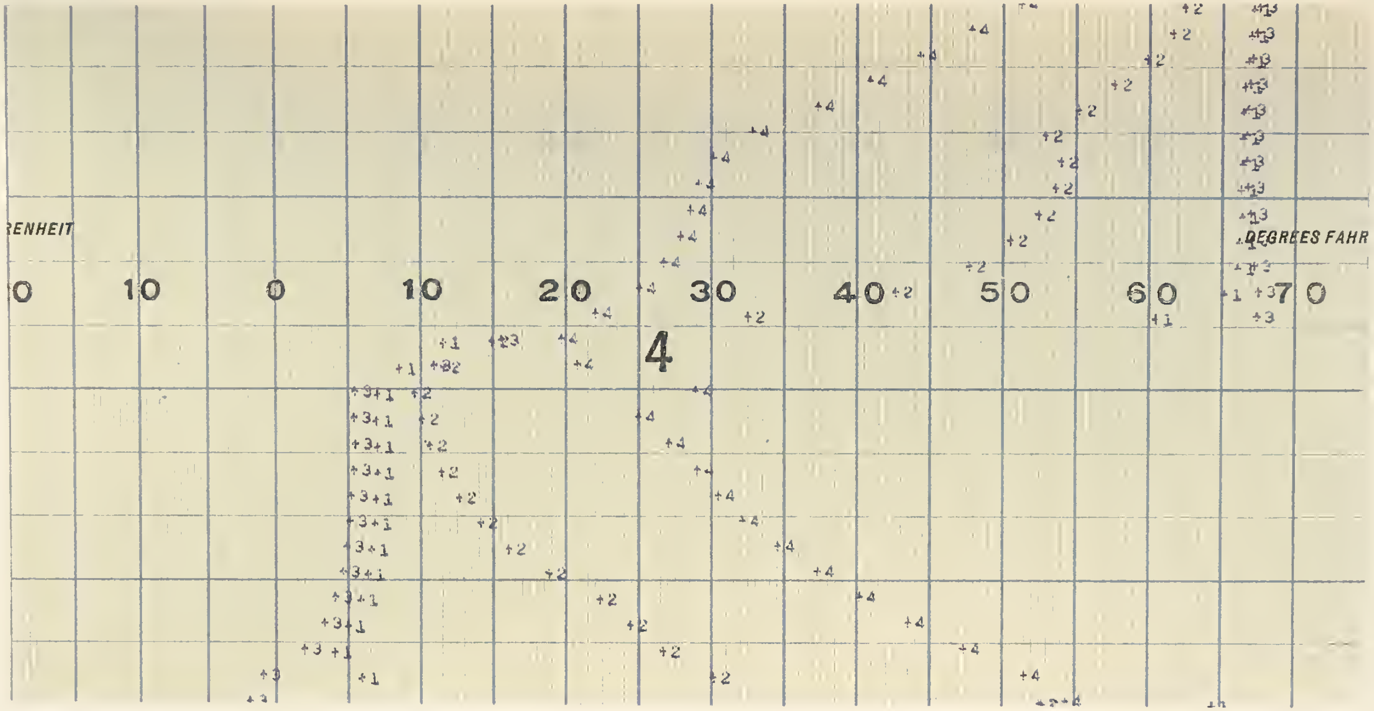


Fig. 58 – Brown Recorder Temperature Records

*2 & *4 – beam temperatures

*1 & *3 – liquid temperatures

Sonic Equipment

Introduction – The deterioration of concrete due to accelerated freeze-thaw tests has been in the past few years determined by the modulus of elasticity which is in turn determined by the frequency of fibration. The equation, derived from the equation of sound and fibration relating the natural frequency of a beam to its modulus of elasticity is:

$$E = \frac{4\pi^2 l^4 w f^2 T}{r^2 m^4 g}$$

where

$E =$ modulus of elasticity, in p.s.i.

$l =$ length of specimen, in inches.

$w/g =$ density of specimen, in $\frac{\text{lbs. sec.}^2}{\text{in}^4}$

$f =$ natural frequency of specimen, in cycles per sec.

$r =$ radius of gyration, in ins.

$= \frac{t}{\sqrt{12}}$ for rectangular cross-section

$m =$ numeric, depending on mode of vibration and on t/l – the ratio of thickness to length.

$=$ 4.73 for fundamental mode, and small ratio of t/l

$T =$ complicated correction term depending on ratio of t/l and on Poisson's Ratio.

Description of Equipment

For a complete description of the sonic equipment and its operation, as well as the procedure used in its operation see "An Investigation of the Air-Entrainment in Concrete and its effect on Durability", a thesis by K. R. Lauer 1948, and "A Preliminary Study of Effects of Air-Entraining Agents, Various Cements, and Curing Conditions on Manufacture of Concrete Specimens", a thesis

by J. L. Jaspar, 1950, Chapter 3, pages 12 – 19. The equipment used was the same as set up and used in the above mentioned theses and as a result no more time will be devoted to a description of this equipment.

APPENDIX III

MISCELLANEOUS TABLES AND GRAPHS

TABLE 15

Approximate Sand and Water Contents per Cubic Yard of Concrete

Based on aggregates of average grading and physical characteristics in mixes having a W/C of about 0.57 by weight or 6½ gallons per sack of cement; 3-in. slump, and natural sand having an F.M. of about 2.75.

Maximum Size of Coarse Aggregates Inches	Rounded Course Aggregate			Angular Coarse Aggregate		
	Sand Per Cent of Total Aggregate by Absolute Volume	Net water Content Per Cubic Yard		Sand Per Cent of Total Aggregate by Absolute Volume	Net Water Content Per Cubic Yard.	
		Pounds	Gallons		Pounds	Gallons
½	51	335	33.5	56	360	36.0
¾	46	310	31.0	51	335	33.5
1	41	300	30.0	46	325	32.5
1 ½	37	280	28.0	42	305	30.5
2	34	265	26.5	39	290	29.0
3	31	250	25.0	36	275	27.5
6	26	220	22.0	31	245	24.5

Adjustments of Values in Above Table for Other Conditions.

Changes in Conditions Stipulated in Table	Effect on Values in Table.	
	Per Cent Sand[1]	Unit Water Content[1]
Each 0.05 increase or decrease in water-cement ratio...........	+or- 1	0
Each 0.1 increase or decrease in F.M. of sand.....	+or- ½	0
Each 1 inch increase or decrease in slump.........	------	+or- 3%
Manufactured sand..................................	+3	+15 lb.
For less workable concrete, as in pavements.......	- 3	- 8 lb.

[1](+) indicates and increase and (-) a decrease corresponding to the conditions stated in the first columns.

(Table #5 and Adjustment of Values for same were taken from the Journal of the American Concrete Institute, November 1943 issue.)

TABLE 16

(From page 131 of Concrete Manual by U.S. Bureau of Roclamation)

Approximate sand and water content per cubic yard of concrete.

Based on rounded aggregates of average grading in mixes having a W/C of about 0.55 by weight, 3-inch slump, and natural sand having an FM of about 2.75.

Maximum size of Coarse Aggregate Inches	Concrete Without Entrained Air			Air-Entrained Concrete[x]			
	Sand % of Total Agg. by Absolute Volume	Net Water Content per cubic yard Pounds	Net Water Content per cubic yard Gallons	Recommended Air Content Percent	Sand % of Total Agg. by Absolute Volume	Net Water Content per cubic yard Pounds	Net Water Content per cubic yard Gallons
1/2	51	335	40	6 ± 1	47	290	35
3/4	46	310	37	5 ± 1	42	270	32
1	41	300	36	4.5 ± 1	37	260	31
1 1/2	37	280	34	4 ± 1	34	245	29
2	34	265	32	4 ± 1	31	235	28
3	31	250	30	3.5 ± 1	28	220	27
6	26	220	26	3 ± 1	24	195	24

Adjustment of Values for Other Conditions

Changes in Conditions Stipulated	Effect on Values: Sand	Effect on Values: Unit Water Content
Each 0.05 increase or decrease in water-cement ratio	± 1%	0
Each 0.1 increase or decrease in FM of sand	±0.5%	0
Each 1-inch increase or decrease in slump	-	± 3%
Each 1-percent increase or decrease in air content	±0.5 to 1.0%	∓ 3%
Each 1-percent increase or decrease in sand content	-	±2.5 lb.
Angular coarse aggregate	+ 3 to 5%	+15 to 25 lb.
Manufactured sand (sharp and angular)	+ 2 to 3%	+10 to 15 lb.
For less workable concrete, as in pavements	- 3%	- 8 lb.

[x] The values listed for air-entrained concrete apply only with Darex or Vinsol resin. Other agents on the market are known to have different reductions for the water requirement.

FIG. 57

DARËX AIR ENTRAINING AGENT REQUIREMENTS - AMOUNTS

(FROM MASTER'S THESIS - K. R. LAUER ,)[6]